THE PRACTICAL DESIGN OF IRRIGATION WORKS

BY

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THIRD EDITION REVISED AND BROUGHT UP TO DATE

BY

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CHIEF ENGINEER IRRIGATION WORKS PUNJAB INDIA [illegible]

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PREFACE TO THE THIRD EDITION

THE second edition of this work was published in the year 1910, and as a good deal of water has flowed down the canals of the world since then—followed by proportionate advance in the science of irrigation engineering—the time is ripe for the revision of the work, bringing it up to date in instructiveness and utility. It is profoundly regrettable that the late Mr. W. G. Bligh did not find occasion to revise his own treatise in a third edition before his death.

In his Preface to the Second Edition Mr. Bligh wrote:

"In this edition considerable emendations and additions have been made to the book, the principal of which is the treatment accorded to the theory of the stability of weirs or other works founded on a sand stratum. In the first edition this stability was made dependent on two considerations—first, the weight of the structure, and secondly, the enforced length of percolation. Exception was taken to this view by professional critics, weight as an agent effecting reduction of pressure, or head, being ruled out as unsound. The author now frankly admits that his previous view is untenable, and Chapter VI. has consequently been entirely recast, the first factor, that of weight, except as regards upward hydrostatic pressure, having been eliminated from the equation of resistance."

Chapter VI., thus recast by Bligh, has become the principal feature of the book, having been made the fundamental criterion of hydraulic design throughout the treatise. The theoretical considerations there set forth have been derived from the record of damages occurring and repairs effected to the weirs across the River Ganges at Narora and the River Chenab at Khanki, on the supposition that the said damages were due to hydrostatic blow-up pressures under the floor of the Narora Weir, and to [illegible] or disruptive percolation flow through the subsoil under the Khanki Weir. The present editor thinks he has conclusively demonstrated in the Notes at end of Chapter VI., that the damages to these Weirs during the period 18[illegible]–1898 were not ascribable to the above-mentioned causes, [illegible] to

by agnostic engineers, in their anxiety to guard their structures, in design, against possible "blow-up" or "piping," during the past quarter of a century The Madras Canal works afford a good pointer, probably, in the direction of sounder, more economical, design

As regards "Retaining Walls" (Chapter I) the editor suggests that the current conventional methods of design and theory are unsatisfactory, and that it is advisable to allow rationally, in calculations of earth-thrust, for the force of soil cohesion, whilst insisting on the rule of the "middle-third" as the only means of designing retaining walls correctly, *qua* foundation soil pressures

In conclusion, the editor desires to make it clear that Bligh's treatise is, in his opinion, an admirable and useful work, all things considered, and a monument to the credit of the industry and acute judgment of its author That does not mean that there is not a great deal in it which the editor is not in full agreement with—especially in Chapters V. and XIII—on which he would not have preferred to express opinions somewhat differently; but it has been considered desirable, as a general rule, not to interfere greatly with the text of the author, and, apart from matters of detail, here and there, to express dissent, if at all, only in editorial footnotes, or in notes appended to each Chapter

F. W. W

LONDON,
March 21st, 1927

PREFACE TO SECOND EDITION

In this edition considerable emendations and additions have been made to the book the principal of which is the treatment accorded to the theory of the stability of weirs or other works founded on a sand stratum In the first edition this stability was made dependent on two considerations—first the weight of the structure and, secondly, the enforced length of percolation Exception was taken to this view by professional critics, weight as an agent effecting reduction of pressure or head being ruled out as unsound The author now frankly admits that his previous view is untenable, and Chapter VI has consequently been entirely recast, the first factor, that of weight, except as regards upward hydrostatic pressure, having been eliminated from the equation of resistance

The next main alteration effected is in Chapter V, where the tables of discharge of submerged weirs founded on theory have been replaced by others worked out from the formula introduced by Herschel an American hydraulician These are based on experimental data which, though on the usual unsatisfactory small scale, still must be preferable to the use of coefficients obtained from any purely theoretical consideration Chapter I remains as before Chapters II and III have been recast and rewritten *de novo* Gravity dams and weirs are now united in Chapter II while the arched and panel deck varieties are dealt with in Chapter III, with many additional examples Chapter IV is little altered Chapters V and VI have already been referred to The remaining chapters have had additional matter inserted and some excised, the most important alterations being in Chapter XII (*olim* XIII), which has been practically rewritten The increase in the number of illustrations all through is considerable

The work has been brought by this means thoroughly up to date While its original characteristics, namely critical examination of existing works, are retained, its scope has been enlarged and its value, it is hoped, as a medium of instruction thereby enhanced

The author wishes to express his obligations to the following works

bearing on the same subject :—"The Irrigation Works of India," R Buckley, C.I.E., London; "Reservoirs for Irrigation Water Power and Domestic Water Supply," James De Schuyler, New York, "Irrigation: Its Principles and Practice," Sir Hanbury Brown, K.C.M.G., London; "Irrigation Engineering," H. M Wilson, New York; "Indian Storage Reservoirs," W. L. Strange, London; "Treatise on Hydraulics," M. Merriman, New York, "Public Water Supplies," F. E. Turneaure and H. L. Russell, New York, "Waterworks Engineering," Tudsbury and Brightmore, London.

W. G. BLIGH.

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LIST OF FORMULAS

CHAPTER II

Section I

No.	Par.	Formula	Subject
1	5	$b = \frac{H}{\sqrt{\rho}} = \frac{2}{3}H$	Base width of dams
2	7	$s = s_1\left(1 \pm \frac{6c}{b}\right)$	Relation of maximum and mean stress
3	10	$W = \frac{H^2 w \sqrt{\rho}}{2} = \frac{H^2}{48}$ Tons	Weight of Δ prism (reservoir empty)
4	10	$R = W\sqrt{\frac{\rho + 1}{\rho}} = 1.2\,W = \frac{H^2}{40}$ Tons	Value of R (reservoir full)
5	10	$s^a = Hw\rho = \frac{H}{16}$ Tons	Maximum unit stress (reservoir empty)
6	10	$s^b = Hw(\rho + 1) = \frac{H}{11.1}$ Tons	Maximum unit stress (reservoir full)
7	10	$H^\lambda = \frac{\lambda}{w(\rho + 1)}$	Limiting depth
8	12	$c = \sqrt{H}$	Crest width
9	13	$b = \frac{H}{\sqrt{\rho}} \times \frac{1}{\sqrt{1 + 2r^2 - 2r^3}}$	Base width of dams when $r = \frac{c}{b}$ in pentagonal profile
10	14	$h = 1.5\sqrt{F} + (2.5 - \sqrt[4]{F})$	Weatherboard

Section II

No.	Par.	Formula	Subject
11	41	$A = H\left(\frac{H + 2d}{2}\right)$	Area of trapezoid of water pressure
11a	41	$A = H^1\left(\frac{H + 2d}{2}\right)$	Area of trapezoid when back of wall is inclined
12	41	$h = \frac{H}{3}\left(\frac{H + 3d}{H + 2d}\right)$	Height of centre of water pressure above base
13	42	$b = \frac{H + d}{\sqrt{\rho}}$	Approximate base width of weirs
14	42	$c = \sqrt{H} + \sqrt{d}$	Crest width of weirs
15	43	$b = \frac{H + d}{\sqrt{\rho}} \times \frac{1}{\sqrt{r + 1 - r^2}}$	Base width when $r = \frac{c}{b}$

NO	PAR	FORMULA	SUBJECT
15a	43	$b = 6\ (H + d)$.	Base width when $r = \cdot 25$ and $\rho = 2\frac{1}{4}$.
16	47	$M = \frac{H^2}{6}(H + 3d)$.	Moment of trapezoid of water pressure
17	49	$Q = 100cD^{\frac{3}{2}}\sqrt{S}$.	Approximate discharge formula of channels.
18	63	$b = \frac{H + d}{\rho}$.	"Hybrid" weir, base width.

CHAPTER III

NO	PAR	FORMULA	SUBJECT
1	3	$b = \frac{RHw}{s_1}$	Base width and mean stress "short" formulas
2	3	λ, or $s = \frac{RHw}{b} \times \frac{2R}{R+r}$.	*Maximum stress.*
3	3	λ, or $s = \frac{2Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)}$.	Maximum stress in terms of R and b. "Long" formula.
4	3	$b = R\left(1 - \sqrt{1 - \frac{2Hw}{\lambda}}\right)$.	"Long" formula for base width.
6	23	$s = R_1 w \rho \sin\ \theta$.	Stress induced by weight of arch ring when inclined.

CHAPTER IV

NO	PAR	FORMULA	SUBJECT
1	13	$P = wrt$.	Horizontal thrust of arch.
2	15	$t = \cdot 2\ r + \cdot 1\ V + 2$.	Thickness of abutments.
3	17	$t = \cdot 4\sqrt{r}$	Thickness of arch.

CHAPTER V

NO	PAR	FORMULA	SUBJECT
1	1	$V = c\sqrt{2gH}$.	Discharge from orifice.
2	1	$Q = cl\ \frac{2}{3}\sqrt{2g}\ (h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}})$.	Discharge from orifice
3	1	$Q = cA\sqrt{2g\ H}$.	Discharge from orifice.
4	3	$Q = cl\ 5\cdot 35\left\{(H\sqrt{H} - h_1\sqrt{h_1}) + \frac{3}{2}\sqrt{H}\ (h_2 - H)\right\}$.	Submerged orifices with velocity of approach.

No	Par	Formula	Subject
5	4	$c_1 = c\sqrt{1 + \frac{h}{H}}$	Modified coefficient
6	4	$Q = cA\sqrt{2g(H + h)}$	Approximate formula orifices
7	10	$Q = \frac{2}{3}\, cl\sqrt{2gd} \times d = 5.35\, cld^{\frac{3}{2}}$.	Free overfall.
8	12	$Q = 3.333\, ld^{\frac{3}{2}}$	Free overfall
8a	16	$q = 3.33\, (nd)^{\frac{3}{2}}$	Submerged fall discharge
9	13	$c_1 = c \times \left\{\left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}}\right\}$.	Modification of coefficient, velocity of approach
13	27	$Q = 100\, Ac\sqrt{RS}$	Discharge of channels
14	27	$C = \dfrac{41.6 + \frac{1.811}{n} + \frac{.00281}{S}}{1 + \left(41.6 + \frac{.00281}{S}\right) - \frac{n}{\sqrt{R}}}$	Kutter's formula
15	31	$V = .705 \times .003\, v^2$	Wagner's mean velocity formula
17	45	$l = \frac{d_2 - d_1}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right) \times \left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right]$	Backwater curve
18	48	$l = -\frac{d_1 - d_2}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right) \times \left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right]$	Falling surface curve.
20	57	$Q = k_1\, 100\, K^{\frac{2}{3}}$	Ryves flood discharge, second feet
21	57	$Q = k\, 100\, K^{\frac{3}{4}}$.	Dickens flood discharge, second feet
22	57	$Q = 200\, K^{\frac{5}{6}}$.	Fanning's flood discharge, second feet.
23	67	$H = M \times \frac{V^2}{2g}$	Head required for syphons
24	67	$V = 8.035\sqrt{\frac{H}{M}}$	Velocity due to head
25	67	$H = M \times \frac{V^2}{2g} - \frac{V_a^2}{2g}$.	Head including velocity of approach
26	67	$V = \sqrt{\frac{2gH + V_a^2}{M}}$.	Velocity including that of approach

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25	67	$H = M \times \frac{V^2}{2g} - \frac{Va^2}{2g}$.	Head including velocity of approach
26	67	$V = \sqrt{\frac{2gH + Va}{M}}$.	Velocity, including that of approach

CHAPTER VI

NO	PAR	FORMULA	SUBJECT
1	21	$W = 4c \times \sqrt{\frac{H^a}{13}}$.	Width of apron. H^a is height of weir shutter crest above floor. In sloping apron H^b, or solid crest above L.W.L., is used.
2	24	$b = \frac{H^t + d}{\sqrt{\rho}}$.	Base thickness of weir wall. H. is height of masonry wall.
3	25	$t = \frac{4}{3} \frac{H - h}{(\rho - 1) \text{ or } \rho}$.	Thickness of apron. H = head, h = neutralised head.
4	27	$L = 10c \sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$.	Distance of talus from crest wall with sloping aprons; $10\frac{1}{2}c$ or $11c$ may be used. q = unit flood discharge.

CHAPTER VII

NO	PAR	FORMULA	SUBJECT
1	13	$L = 15c \sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$.	Distance of toe of talus from head work.
2	13	$W = 7c \sqrt{\frac{H_a}{13}}$.	Width of apron.
3	30	$t = \sqrt{\frac{3H}{2}}$.	Thickness of floor.

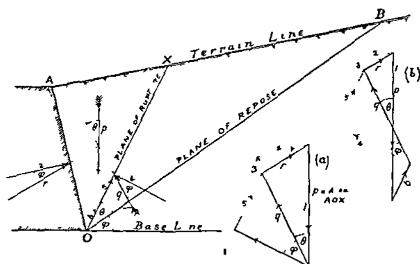
SUMMARY OF TABLES IN CHAPTER V.

THE PRACTICAL DESIGN OF IRRIGATION WORKS

CHAPTER I

RETAINING WALLS

(1) RETAINING WALLS whose function is the retaining or holding back of earthen banks form one of the most considerable items of the component parts of masonry structures The design of such should satisfy conditions that combine economy of material with stability To effect this desirable



FIGS 1 1a 1b

end with accuracy and in accordance with scientific principles it will be necessary to possess a thorough understanding of the theoretical problem involved

(2) The diagram in Fig 1 presents a graphical statement of the theory of earth pressure AO is the back of the retaining wall AB the terrain line, or top surface of the bank which is upheld and OO_1 the horizontal base line *representing the natural surface of the ground* OB *is the plane of* repose of the earth forming this backing the angle BOO_1 or ϕ being thus the angle of repose *i e* the steepest slope at which the earth will stand

when unsupported The triangle AOB represents, therefore, the largest prism of earth held up by the wall

In case of failure of the wall, the earth backing will break away in some line OX in the prism AOB, intermediate between AO and OB, and OX is termed the plane of rupture Of all possible positions of OX, that one which ensures the maximum thrust on the wall has alone to be considered as the true plane of rupture, and thus the prism AOX is the portion of the earth backing tending to overturn the wall and will be found, as hereafter explained to have a much greater effect than the larger whole prism AOB, on the supposition that the plane of rupture is coincident with that of repose, which is not the case

(3) By reference to Figs 1a and 1b the forces in equilibrium will be found as follows —

(1) The weight of the prism AOX acting vertically
(2) The normal reaction of the retaining wall acting at right angles to the direction of the back of the wall OA
(3) The friction of the prism on the inner surface of the retaining wall acting upwards
(4) The normal reaction of the plane of rupture OX
(5) The friction of the prism AOX on the plane of rupture acting upwards
(6) The cohesion of the earth in the plane of rupture acting upwards

(4) Fig 1a represents the force polygon of forces 1 to 5, Fig 1b the same but inclusive of (6) In both diagrams forces 2 and 3 are components of r and 4 and 5 of q The polygon is thus resolved into the triangle of forces $p\ q\ r$ Of these p is the weight of the prism AOX, represented by the area of the triangle AOX, q is the reaction of the plane of rupture, and r the resultant pressure on the back of the wall, both q and r being modified in direction by angles of friction The direction of q is determined by the angle θ, which it forms with the vertical p, the length of r is dependent on the value of this angle θ Now θ is also the angle between the planes of repose and of rupture or the divergence of OB and OX Hence the primary necessity of ascertaining the true value of this angle, on which that of the force r is dependent

(5) The two diagrams 1a and 1b, show clearly the great reducing influence exerted on the value of r by the introduction of the comparatively small force (6) representing cohesion No definite value can possibly * be assigned to cohesion as it must necessarily vary with differing soils and conditions of moisture consolidation etc, for instance from pure sand which is almost devoid of the property of cohesion, to stiff clay in which this property is highly developed Consequently we are compelled to omit

* But see Editor s Notes at the end of this Chapter —ED

this force, though always present in a greater or less degree, from inclusion in the graphical statement

Although this omission will apparently somewhat vitiate the value of the theory of earth pressure, as it presupposes a state of things in the condition of the earth which does not actually exist, yet it will be found that the neglect of cohesion will not preclude the successful use of the remaining five forces to which approximately correct values can be assigned provided that the unknown value of cohesion be considered as supplying a definite, though not precisely ascertainable factor of safety in favour of the stability of the wall

It is well known that retaining walls, which according to received theory are designed of too weak a section to be safe are in reality possessed of a large factor of safety This point was fully brought out in the discussion on retaining walls in the "Minutes of the Proceedings of the Institution of Civil Engineers," Vol LXV Consequently the theory of earth pressure, though not reducible to absolute exactitude as in the case of fluid pressure, will still be found an indispensable and reliable guide in the economical design of retaining walls The sections need only be designed of sufficient statical properties to withstand the theoretical earth pressure That is to say, the moment of stability of the wall need be only just equal to that of the aggressive forces, or in equivalent graphical terms, the resultant centre of pressure of the combined forces can be allowed to pass just within the outer toe of the base of the section, leaving no factor of safety beyond that supplied by the unknown influence of cohesion and further designs based on a slight modification of this principle will be found in agreement with commonly received and established practice

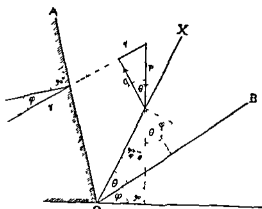


FIG 2

(6) Fig 2 illustrates more fully the triangle of forces p q r with their angles of divergence The resultant r it will be seen forms an angle with the normal to OA of the assumed value of ϕ^1 tan ϕ^1 being the coefficient of friction of earth against the wall This force, which acts in favour of the wall, causing a downward deflection in the direction of r, is neglected by some theorists but without tangible reason It is evident that friction must develop on this surface, as well as on the plane of rupture on any movement of the wedge of earth AOX consequently it cannot be ignored The value of ϕ^1 is, however, not considered as equal to ϕ and in Chalmers Graphical Determination of the Stresses in the Structures of Engineering the value of ϕ^1 is assessed at one half that of ϕ This proportional value will be adopted * As regards ϕ, a consensus of authorities place the slope of repose

* But see Editor s Notes at the end of this Chapter —ED

at $1\frac{1}{2}$ to 1 which is what ordinary soil naturally assumes Some clays however have a much flatter slope of repose as 2 or 3 to 1 in such cases however the backing should be formed of prepared material which will not have a flatter angle of repose than $1\frac{1}{2}$ to 1 Sir Benjamin Baker in the paper on the pressure of earth on retaining walls above referred to assumes $1\frac{1}{2}$ to 1 as the slope of repose although dealing with London clay with a natural angle of 3 to 1

(7) The earth backing is thus theoretically considered as a fluid having the specific gravity of earth but subject to modifications by the influence of friction on the surfaces on which it slides The reactions of the back of the wall and of the plane of rupture in accordance with the properties of fluids must be normal to those surfaces but the action of friction causes obliquity n the directions of q and of r from the lines normal to OA and OX

(8) In all subsequent investigations the value of ϕ will be assumed at 1^1 to 1 ϕ^1 at 3 to 1 With regard to p the weight of earth will be taken as 1 cwt per cubic foot or specific gravity 1 8 Some clays weigh as much as 120 lbs or 125 lbs per cubic foot or have a specific gravity of 1 9 or 2 0 but they are exceptional and can be met by special design In hot countries where alone irrigation is practised the heavy clays common in Europe which are due to glacial action are rarely met with Most soils do not weigh over 100 lbs per cubic foot

Position of the Plane of Rupture

(9) It has already been observed (*vide* par 2) that the position of OX is somewhere intermediate between OB and OA By reference to Fig 2 it will be seen that if the plane of rupture be assumed to coincide with that of repose the triangle AOX will be coincident with AOB and consequently p will be at a maximum value The angle θ however disappears and with it r On the other hand were OX coincident with OA p would disappear Some intermediate position of OX must be found which will satisfy the condition of giving a greater value to r than any other possible one In works where this subject is treated by analytical methods a formula is given whereby the angle θ can be found by calculation but it is so involved as to be useless for practical purposes In Chalmers work the problem is solved by graphical process but except in the case of a horizontal terrain line the system adopted is too abstruse and complicated to be of practical value though doubtless suitable for students as an exercise of ingenuity The process adopted in this work for finding the correct value of θ where the terrain line is not horizontal i [illegible] ntal in method and as simple as possible Various positions of [illegible] rupture are tentatively assumed the value of r deduced in eac[illegible] t inclination giving the greatest value to r is adopted as the co[illegible] θ

(10) When the terrain line AB [illegible] orizontal and the angle of friction is neglected OX will bisect the angle AOB The proof of this can be found

in any work on the subject When the friction on AO is taken into account the direction of r will be more inclined and thus will tend to slightly modify the position of OX but to so small an extent that for all practical purposes it may be entirely neglected

(11) In cases where the terrain line AB is inclined upwards or downwards as shown in Figs 3 and 4 the position of OX has to be found by a tentative process as exhibited below

Fig 3 is a case with a terrain line inclined upwards at a slope of 2 to 1 On AB any number of points X^1 X^2 etc are taken these joined by the dotted lines with O form so many planes of rupture each having different

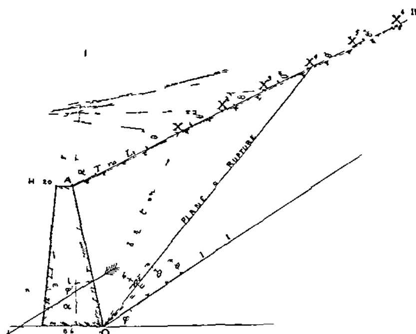


FIG 3

values of θ viz the angles X^1OB X^2OB etc We have now to find which of these directions will ensure the greatest value of R the resultant pressure on the wall As the triangles AOX^1 AOX^2 etc have the common base AO their areas are directly proportional to their several sides AX^1 AX^2 etc consequently these sides can be taken as representative of their areas *i e* of p^1 p^2 etc the weights of the prisms held up by the wall

To avoid constructing a separate figure the force polygon can be joined on to the diagram by assuming AX as the vertical load line At the point A the direction of r is set off making an angle with p equal α on the figure in the same way at the points X^1 X^2 etc the angles θ^1 θ^2 which OX^1 and OX^2 make with OB are set off toward the line r These lines which represent the oblique reactions of the planes of rupture cut r in several places These several intercepts of the line above the point A give the value of r for each trial plane of rupture and that angle effecting the largest intercept is

at $1\frac{1}{2}$ to 1 which is what ordinary soil naturally assumes Some clays, however, have a much flatter slope of repose, as 2 or 3 to 1, in such cases, however the backing should be formed of prepared material, which will not have a flatter angle of repose than $1\frac{1}{2}$ to 1 Sir Benjamin Baker, in the paper on the pressure of earth on retaining walls above referred to, assumes $1\frac{1}{2}$ to 1 as the slope of repose, although dealing with London clay with a natural angle of 3 to 1

(7) The earth backing is thus theoretically considered as a fluid, having the specific gravity of earth but subject to modifications by the influence of friction on the surfaces on which it slides The reactions of the back of the wall and of the plane of rupture, in accordance with the properties of fluids, must be normal to those surfaces, but the action of friction causes obliquity in the directions of q and of r from the lines normal to OA and OX

(8) In all subsequent investigations the value of ϕ will be assumed at $1\frac{1}{2}$ to 1 ϕ^1 at 3 to 1 With regard to p, the weight of earth will be taken as 1 cwt per cubic foot or specific gravity 1 8 Some clays weigh as much as 120 lbs or 125 lbs per cubic foot, or have a specific gravity of 1 9 or 2 0, but they are exceptional and can be met by special design In hot countries, where alone irrigation is practised the heavy clays common in Europe, which are due to glacial action are rarely met with Most soils do not weigh over 100 lbs per cubic foot

Position of the Plane of Rupture

(9) It has already been observed (*vide* par 2) that the position of OX is somewhere intermediate between OB and OA By reference to Fig 2, it will be seen that if the plane of rupture be assumed to coincide with that of repose the triangle AOX will be coincident with AOB, and consequently p will be at a maximum value The angle θ, however, disappears, and with it r On the other hand, were OX coincident with OA, p would disappear Some intermediate position of OX must be found which will satisfy the condition of giving a greater value to r than any other possible one In works where this subject is treated by analytical methods a formula is given whereby the angle θ can be found by calculation, but it is so involved as to be useless for practical purposes In Chalmers' work the problem is solved by graphical process, but except in the case of a horizontal terrain line, the system adopted is too abstruse and complicated to be of practical value, though doubtless suitable for students as an exercise of ingenuity The process adopted in this work for finding the correct value of θ, where the terrain line is not horizontal is [illegible] method and as simple as possible Various positions of [illegible] tentatively assumed the value of r deduced in eac[illegible] [illegible]ing the greatest value to r is adopted as the co[illegible]

(10) When the terrain line A[illegible] [illegible]ction is neglected, OX will bisect the angle [illegible] [illegible]nd

earth and water acting on the wall, as if composed of the former material

The slight excess in the area of the superimposed parallelopiped AX^1, which is somewhat larger than the rectangle A^1X, is too trifling for consideration, though capable of exact adjustment by further lowering the line A^1X

(14) Process of drawing resultant line of pressure to base of wall profile

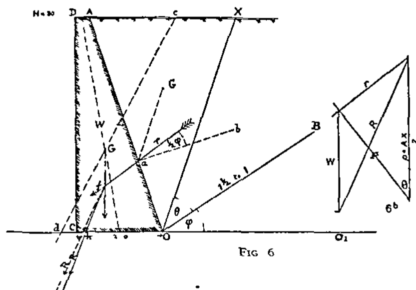


FIG 6

Fig 6 represents a trapezoidal section of wall 30 feet high 2 feet wide at crest, and 12 feet at base The face is vertical The object of drawing the resultant line of pressure through the base of the wall is that the position of the point where this intersects the base forms the criterion of the stability of the section adopted If it falls at the outer toe, the section is of just sufficient statical stability, if outside, the wall will overturn *The further the point of intersection falls within the outer toe the greater* the stability Excess, however, would lead to waste of material As already noticed in par 5 the unknown force of cohesion forms a large but indefinite factor of safety It is however, deemed advisable to ensure a certain small additional precise factor by designing the section so that the resultant line of pressure will intersect the base within the outer toe at a distance having a certain arbitrary proportion to the width of base This will obviate * any remote possibility of crushing the material at the edge of the base, which would be liable to occur were the maximum point of pressure concentrated at this

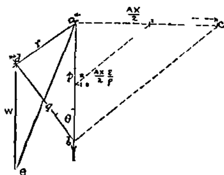


FIG 6a

* This cannot be decided unless the stress diagram be drawn to show maximum pressure intensity at toe of wall —ED

point This inset being a fraction of the base will ensure a uniform system of design applicable to all sections and one which is in agreement with practical examples of known stability the weight of these trapezoidal walls thus bearing a constant ratio to the base width b the top width being a fixed dimension

This fraction will be fixed at one eighth the width of base and has been decided on with due regard to assimilation of design to the proportions of retaining walls such as are commonly followed in practice The problem of design now consists in adjusting the base width of the sections so that the resultant line of pressure will cut the base at a point $\frac{b}{8}$ within the toe This can only be effected by a tentative process of trial and error As the section is determined entirely by the base width any alteration in the latter affects the back slope of the wall and with it the oblique direction of r which is at a fixed angle from the normal to the back furthermore the area of the triangle of earth pressure the position of OX and value of θ are all modified by any alteration in the base width of the profile of the wall

To express this in analytical terms would necessitate the use of formulas of so complicated a form as to prove too cumbersome by far for practical purposes and the time spent in working them out would greatly exceed that employed in making the graphical solution besides the increased liability to error

(15) As already noted in par 8 the specific gravity of earth backing is taken as 1 8 for these investigations With regard to the walls carefully built rubble masonry will weigh from 140 lbs to 145 lbs per cubic foot *i e* have a specific gravity of $2\frac{1}{4}$ This value is adopted in Water Works Engineering * as applicable to masonry dams where it is stated that the specific gravity of the stone used with careful construction should in cases of walls subject to water pressure bring the weight of the built masonry to not less than 145 lbs per cubic foot

For retaining walls of ordinary construction a specific gravity of $2\frac{1}{4}$ is deemed too high and in this work 2 1 will be adopted for earth retaining walls *and 2 25 for stone dams and weirs*

Granite masonry has a specific gravity of $2\frac{1}{2}$ or even 3 or more with Portland cement mortar Brickwork varies from 100 lbs to 125 lbs per cubic foot *i e* from specific gravities 1 6 to 2 0 In these investigations 1 8 will be adopted as the specific gravity of brickwork The following is a *resume* of the foregoing —

Earth Backing Specific Gravity	1 8
Brickwork Specific Gravity	1 7 to 1 8 in lime mortar
Ordinary Stone Masonry	2 1
Special	2 25
Granite	2 5 to 3 in cement
ϕ	$1\frac{1}{2}$ to 1
ϕ^1	3 to 1

* Water Works Engineering by Tudsbery and Brightmore

(16) To revert to Fig 6 OB is the plane of repose, set out at 1½ to 1 from the base line OO_1 The terrain line AX being horizontal the plane of rupture bisects the angle AOB, and $\theta = \frac{AOB}{2}$ The prism of earth AOX, which exerts active pressure on the wall will do so by sliding on its base OX The resultant line of pressure of this weight will act through the centre of gravity of the triangle AOX, and a line shown dotted on figure drawn parallel to the base of the triangle AOX through G cuts the back of the wall AO at the point marked a Oa is clearly one-third of AO, and thus we see that whenever the apex of the triangle of earth pressure is coincident with the inner point of crest of wall the resultant earth pressure invariably acts at one-third the vertical height of wall above base or at $\frac{H}{3}$. In Fig 5 the apex of the corresponding triangle is not at A but at A^1 higher up, consequently the centre of pressure will be at a greater height above the base

(17) Having found a the point of application of r its direction is found by drawing ab at right angles to AO and from it setting off r upwards at an inclination of ϕ^1 or 3 to 1 from the normal ab The line r should be continued through the profile This force will now have to be combined with that consisting of the weight of the masonry acting through its centre of gravity

By far the easiest method of finding the centre of gravity of a trapezoidal figure is as follows —From A measure off Ac = base OC and similarly from C, Cd=crest AD join cd and also the points of bisection of AD and OC The intersection of these lines is G the centre of gravity of the profile $ADCO$ From this point G draw a vertical cutting r at the point f

The directions of the two forces r and W are now connected on the profile and the next step is to find out their values

Let H denote the vertical height of the wall Then the weight of the prism AOX, taken as one unit wide, is $\frac{H}{2} \times AX \times w$ w being the weight of a cubic foot of the earth backing Similarly that of the wall will be $H \times W \times w_1$ W being the mean thickness of the wall Let ϵ be the specific gravity of earth ρ that of masonry then the weight of a cubic foot of water being 62 3 lbs $w = \epsilon \times 62.3$ and $w_1 = \rho \times 62.3$ Hence the two expressions which have to be graphically equated become

$$H \times \frac{AX}{2} \times \epsilon \times 62.3$$

$$H \times W \times \rho \times 62.3$$

eliminating the common factors we have as representative of the loads on either side

$$\text{Earth} = \frac{AX}{2} \times \epsilon$$

$$\text{Masonry} = W\rho$$

Reducing the earth expression to the base of the specific gravity of masonry, i e, by dividing both expressions by ρ, we have

$$\text{Vertical earth pressure} = \frac{AX}{2} \times \frac{\epsilon}{\rho}$$

$$\text{Vertical wall pressure} = W$$

(18) This arrangement for the simplification of expressions is commonly adopted in graphical calculations

The force polygon is shown in Fig 6a Here the vertical force is represented by $ab = \frac{Ax}{2} \times \frac{\epsilon}{\rho}$ To effect this fractional multiplication graphically, ac is drawn horizontally $= \frac{AX}{2}$ On ab mark off on any scale, 1 8 the specific gravity of earth and on ac, 2 1, that of masonry, join these points and through c draw cb parallel to the last line Then by similarity of triangles, ab or p will $= \frac{ac \times 1\,8}{2\,1} = \frac{AX}{2} \times \frac{\epsilon}{\rho}$ From b set off bd inclined at the angle θ from p, and through a draw ad parallel to the line r in the profile Fig 6 The triangle abd will then represent the forces p q and r Having now obtained r in quantity as well as direction, it has to be combined with the force W This can be best effected on the same diagram The procedure is as follows —Draw de vertically $= W$, i e $= 7$ ft and join ae The triangle ade will then be the force triangle required, ad being r and de W, the resultant of these two forces R will be ae, the third side of the triangle ade, in direction and magnitude R is then the resultant line of pressure on the base which is required In Fig 6 from the point f draw R parallel to ae in Fig 6a cutting the base at the point h

(19) The distance of h from the outer toe C will be found to be close to 1 foot 6 inches * or $\frac{1}{8}$ base CO When the wall is of brickwork of the specific gravity 1 8 the same as that of the earth backing, the force p is drawn equal to $\frac{Ax}{2}$ This is shown in Fig 6b The resultant R_1 reciprocal to that in Fig 6b is drawn as a dotted line on the profile Fig 6 Its incidence on the base CO is nearer the toe C than that of R To bring the incidence of R_1 back to the point h, which latter is at $\frac{b}{8}$ distance from C, the base width will have to be increased by half a foot and be in terms of H, $4H + \frac{1}{2}$, instead of $4H$, as it stands in the drawing

The Reciprocal Triangle of Earth Pressure

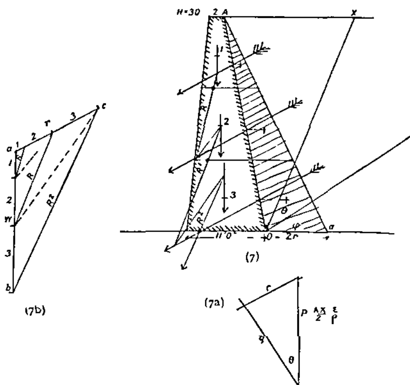
(20) Hitherto we have only been concerned with the point of intersection of all the forces engaged on the base line of the retaining wall the position of which point demonstrates the stability of the section and which is

* Actually 1 25 foot or b/10 —ED

identical in result with the analytical system of taking moments about the outer toe, or some other fixed point in the base line

This procedure is sufficient in most cases, as it can be safely assumed that if the wall at the weakest part, viz, the base, is in stable equilibrium, the upper part will be still more so When, however, retaining walls are of exceptional height, or are subjected to abnormal conditions of pressure, or are of unusual section, it will become desirable that the line of pressure be traced right through the section of wall from crest to base, thus enabling the designer by inspection of the weak points to modify the profile as may be expedient

(21) In order to be in a position to draw the line of pressure, the distribution of the earth pressure on the back of the wall has to be ascertained, the



Figs 7, 7a and 7b

previous method only giving the point of application of the resultant on the base

To effect this the triangle of earth pressure reciprocal to that of the earth pressing on the wall has to be drawn reduced to a masonry base, so that ordinates drawn parallel to the base of this triangle to any point on to the back of the wall will truly measure the intensity of pressure at such point

In Fig 7 the triangle *AOX* is the area of earth backing whose weight presses on the wall The pressure developed by this prism is *nil* at the

crest A and is a maximum at the base, hence the form of the area of pressure will be that of a triangle having its apex at the crest A. The area of this triangle should equal the total earth pressure, and each ordinate drawn parallel to its base will measure the intensity of unit pressure at that point. Now the total earth pressure in terms of unit weight of masonry is, as we have already seen, rH, r, as in Fig. 7a being the resultant of the earth pressure divided by H (the weight being unity), thus the area of the triangle must $= rH$, *i.e.*, its mean width will be r and its base will measure $2r$. In Fig. 7, if Oa is made $= 2r$ then the triangle AOa will be the reciprocal triangle of earth pressure, its area being rH. The base Oa being horizontal, the measure of the intensity of unit pressure at any point on the back of the

Fig. 8

wall AO will be the horizontal ordinate of the triangle AOa. The actual direction of the pressure however, is oblique, being inclined to the normal to the back AO.

(22) The procedure of drawing the line of pressure will be briefly described. The wall and triangle of pressure are divided into laminas of equal depth, in this case three. Their common depth $\frac{H}{3}$ can then be eliminated from the graphical calculation, the weights of the laminas can then be represented in the force diagram 7b by their respective mean widths.

Thus in Fig. 7b the load line ab or W is composed of the three half widths of the wall 1, 2 and 3. In the same way ac or r is composed of the three half widths, 1^1, 2^1 and 3^1, of the laminas of the triangle of pressure. The incidences of r^1, r^2 and r^3 on the back of the wall are at the intersection

of lines drawn through the CG's of the laminas From these points the directions of the forces are inclined parallel to r in Figs 7a and 7b From the intersection of 1^1 with 1 in Fig 7, R is drawn parallel to its reciprocal R in 7b, the point where this cuts the end of the base of the lamina 1 is a point on the line of pressure From the intersection of R continued, with 2^1, a line is drawn back parallel to its reciprocal in Fig 7b, the resultant of the three forces 1, 1^1 and 2^1 Again from its intersection with the force 2 the line R^1 is drawn to meet the inclined force 3^1 parallel to its reciprocal R^1 in the force diagram, this intersects the base of lamina 2, giving a second point in the line of pressure and terminates as before, at its junction with the inclined force 3^1 The procedure as above described with regard to the reverse line gives the starting point of the final resultant R^2 which intersects the base of the wall This latter point is clearly identical with that obtained by the simpler procedure formerly given

(23) We will now proceed to show the method of obtaining the incidence of the resultant line of pressure on the base of a trapezoidal wall in cases where the terrain line is not horizontal and also where it is surcharged with a weight above the level of the wall crest

Fig 8 represents a case with the terrain inclined upwards First the point X, or the intersection of the line of rupture with the terrain, is obtained by the means described in par 11 The area of the prism of earth pressing on the rear of the wall AOX cannot be represented as was done hitherto by $\frac{AX}{2} \times H$, but if the line XX^1 be drawn parallel to the back of the wall till it intersects the horizontal line AX^1 and the point X^1 joined with O, then the triangle AOX^1 is evidently equal to AOX and its area is equal to $AX^1 \times H$

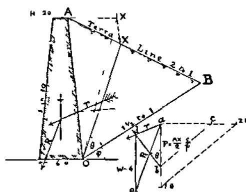


Fig 9

The centre of gravity of the prism is evidently on a line drawn parallel to the base OX consequently the incidence of the inclined force r on the back of the wall will be at one third of H The same is the case with any triangle having its apex at the point A The procedure of finding r is identical with that already described, p being made equal to $\frac{AX^1}{2} \times \frac{\epsilon}{\rho}$, and q set off at the angle θ from its extremity b, which intercepts the inclined line r drawn from the origin a

(24) In Fig 9 we have a similar case but with the terrain line inclined downward The procedure is identical with that in the last case

(25) Fig 10 is a replica of Fig 5, in which the terrain line is loaded with water As already noticed in par 13 the prism of earth acting on the wall

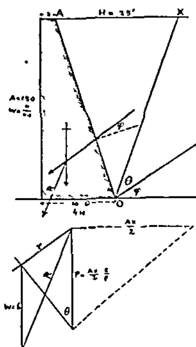
is the triangle A^1OX^1 Consequently in the force polygon Fig 10a p is made either by graphical or arithmetical process equal to $\frac{A^1X^1}{2} \times \frac{\epsilon}{\rho} \times \frac{H^1}{H}$ The last fraction enables H to be eliminated as a common factor By setting off q at the angle θ from the base of the load line the value of r is obtained, as also the base Oa of the triangle of pressure AOa which is $2r$ The earth pressure is represented not by the triangle A^1Oa but by the hatched trapezium AOa consequently r, in Fig 10, is not representative of the mean pressure but r^1 the half-width of the trapezium which is greater than r does so Hence in Fig 10a r^1 has to be measured off from the point of origin a to d and the vertical W or de is drawn through this point d equal to the half width of the trapezoidal wall ae is then the direction of R In the profile Fig 10 the inclined force r^1 cuts the back of the wall at the inter-

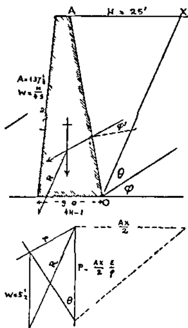
Figs 10 10a

section of a horizontal through the centre of gravity of the trapezium AOa, not at that of the triangle A^1Oa

(26) The consideration of further abnormal conditions to which retaining walls are subject will now be deferred until the principles of design are worked out in regard to trapezoidal walls of various heights and face batters under ordinary conditions of earth pressure For this purpose Figs 11 to 15 have been prepared being a series of diagrams of sections of equal strength but of varying face batters placed side by side for purposes of comparison Inspection of these diagrams will at once demonstrate to the eye the influence exerted in the economical design of the sections by face batter, the diagrams also furnishing reliable types of sections for practical use based on which formulas easy of application can be deduced

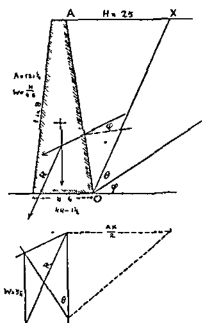
In all cases the specific gravity of the material of the wall is taken as 2 1, that of the earth backing as 1 8 the top width at crest is a fixed dimension of 2 feet, and each section is so designed that the centre of pressure falls close to the outer toe at a distance of one eighth of the base from that point As noted in par 19 if the wall is of brickwork with a specific gravity the same

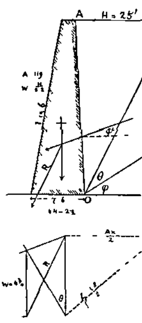

Fig 11

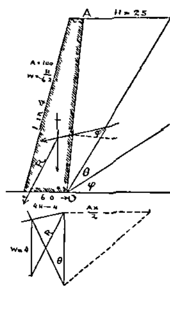

Fig 12

as the earth backing, the incidence of the resultant on the base will be beyond $\frac{b}{8}$ from the toe at a point about $\frac{b}{10}$* from the same point

Thus brick walls built of these proportions will be safe though of less


Fig. 13.


Fig 14


Fig 15

* But see foot note to par (19) The fraction will be less than b/10 in this case —Ed

stability or if the same statical condition as was deemed requisite in the case of stone walls be insisted on the base should be increased from 6 inches in the higher to 4 inches in the lower sections right through

TABLE I

Face	H	Base	Area square feet	$\frac{H}{W}$	Height (H) when back becomes vertical	Face	H	Base.	Area square feet	$\frac{H}{W}$	Height (H) when back becomes vertical
	Feet				Feet.						Feet
Vertical	30	$4H$	210	$\frac{1}{4\ 3}$	5	Batter 1 in 10	30	$4H—1$	195	$\frac{1}{4\ 6}$	10
	25	$4H$	150	$\frac{1}{4\ 2}$			25	$4H—1$	137⅓	$\frac{1}{4\ 5}$	
	20	$4H$	100	$\frac{1}{4}$			20	$4H—1$	90	$\frac{1}{4\ 4}$	
	15	$4H$	60	$\frac{1}{3\ 8}$			15	$4H—1$	52½	$\frac{1}{4\ 3}$	
	10	$4H$	27½	$\frac{1}{3\ 4}$			10	$4H—1$	25	$\frac{1}{4}$	
Batter 1 in 8	30	$4H—1\frac{1}{2}$	187	$\frac{1}{4\ 8}$	12 8	Batter 1 in 6	30	$4H—$[illegible]⅓	172½	$\frac{1}{5\ 2}$	17 3
	25	$4H—1\frac{1}{3}$	131	$\frac{1}{4\ 8}$			25	$4H—2^1$	119	$\frac{1}{5\ 2}$	
	20	$4H—1\frac{1}{2}$	85	$\frac{1}{4\ 7}$			20	$4H—2\frac{1}{2}$	75	$\frac{1}{5\ 2}$	
	15	$4H—1^1$	49	$\frac{1}{4\ 6}$			15	$4H—2\frac{1}{4}$	43	$\frac{1}{5\ 4}$	
	10	$4H—1\frac{1}{2}$	22½	$\frac{1}{4\ 4}$			10	$4H—2$	20	$\frac{1}{5}$	
Batter 1 in 4	30	$4H—4$	150	$\frac{1}{6}$	Back is sloped inward throughout						
	425	$4H—4$	100	$\frac{1}{6\ 2}$							
	20	$4H—4$	60	$\frac{1}{6\ 7}$							
	15	$4H—3$	38	$\frac{1}{6}$							

NOTE —H = vertical height W = mean width of walls (both in feet) $\rho = 2\ 1$

Reference to the diagram series will show that with the change of face batter the consequent gradual reduction of the base width causes the back slope to become steeper till at last in Fig 15 it vanishes and the back slope becomes inward in sense This diminishing of the base is so regular and so clearly governed by the height that the base width can well be expressed in

terms of H, and rules can be framed giving the required base width of any wall as a fraction of its height In this the base for a vertically faced wall, as in Fig 11, will be used as a standard, the whole of that series working out to $4H$ The base width of Fig 12 works out to $4H—1$, of Fig 13 to $4H—1\frac{1}{2}$ of Fig 14 to $4H—2\frac{1}{2}$ and of Fig 15 to $4H—4$

Sections have been drawn out for every depth from 30 to 10 feet for each face slope, though not given here, and the tabular statement on p 16 is the result

(27) In practice many revetment walls as wings of bridges have a crest sloping down at $1\frac{1}{2}$ or 2 to 1 in uniformity with the side slope of an embankment which they support If designed of equal strength throughout, in accordance with the base widths in the Table the end portion of the wall in some cases would have an inward and the upper an outward back slope Although it is possible to build the wall in this way the outer slope gradually *changing from inward slope to vertical and again outward yet it would be a* somewhat awkward construction, and for the sake of simplicity it would be as well to continue the wall as soon as the outer slope is merged into the vertical as one with a vertical back this will cause the lower portion of the wall to have an excess of strength and material This fact would militate against the use of excessive batter as 1 in 4 or even 1 in 6 for crest sloping walls as the saving effected in the higher portion would be discounted by the excess in the lower In the tabular statement given above the height at which the back slope disappears and the back becomes vertical is noted in the last column of the Table

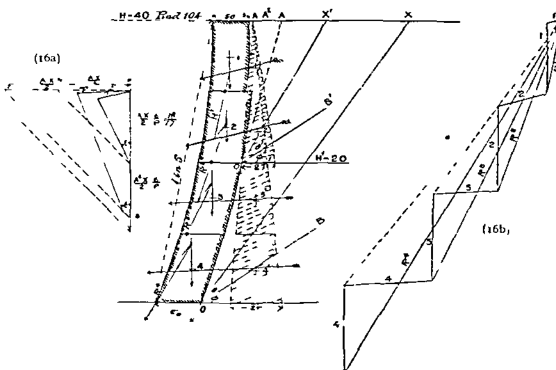
(28) The great economical advantage of face batter to retaining walls is clearly proved by the sections given and by the Table For instance the vertically faced wall in Fig 11 has a sectional area of 150 square feet and a proportion of $\frac{W}{H}$ or $\frac{\text{mean width}}{\text{height}}$ of $\frac{1}{4\,2}$ Whereas in Fig 15 of equal strength the area is 100 square feet and the fraction $\frac{W}{H}$ is $\frac{1}{6\,2}$ In this case the back of the wall has an inward slope thus approximating to a pitched slope

(29) Brunel s curved walls had a value of $\frac{W}{H}$ of nearly one seventh and leaned over to that extent that the centre of gravity of the wall fell outside the heel of the base A specimen of the section of one of these retaining walls is given in Fig 16

For the graphical calculation the back of the wall is considered as consisting of two planes A^1O^1 and O^1O O^1 being at half the height or 20 feet above the base As these portions of the back have different inclinations to the vertical the directions of their several planes of rupture and values of θ and r will also vary Thus a separate graphical calculation will have to be made for each of the portions viz the whole AO and the part A^1O^1

The lower portion of the back OO^1 is continued up to meet the terrain line

at A The planes of repose OB and O^1B^1 are then set out at $1\frac{1}{2}$ to 1 with the horizontals from O and O^1, and the planes of rupture OX and O^1X^1 are found by bisecting the angles AOB and $A^1O^1B^1$ The angles θ and θ^1, are obtained by this construction The area of earth pressing on the whole wall is the four-sided figure A^1O^1OX, or the triangle AOX + the triangle AO^1A^1. If the apex A be moved to A^2, so that $AA^2 = AA^1 \times \frac{H^1}{H}$, then the enlarged triangle A^2OX will equal the combined area of the two triangles just noted. H^1 being $= \frac{H}{2}$, the position of A^2 will be at the bisection of AA^1 Hence for



FIGS 16 16a, 16b

purposes of measuring the area of the earth triangle, A^2X will be considered as the base in lieu of AX

With regard to the upper portion, the triangle $A^1O^1X^1$ represents the earth prism pressing on the upper half back A^1O^1, A^1X^1 will then measure the comparative weight of this part

(30) To find the values of r for each division the force triangle 16a is now constructed The vertical load line is made $= \frac{A^2X}{2}$, for the whole depth of wall, and on the same line the value of p for the upper half, viz, $\frac{A^1X^1}{2}$, is marked off From the extremity of the load line and also from the higher point in in the angles θ and θ^1 are set off, intercepting r and r^1 on the two

lines representing the directions of these forces, which lines are inclined at angle ϕ, above the normal to the planes AO and A^1O^1 respectively

The next step is to construct the reciprocal triangles of earth pressure With regard to the upper portion, $2r^1$, drawn horizontally from the point O^1, will form the base of the triangle, its apex being at A^1 (*vide* par 21)

The lower base could in similar manner be measured $= 2r$ from the point O, the apex of this larger triangle of pressure being at A^2 It will be more convenient, however, to make the inner side of this triangle coincide with that of the upper, viz, with the line A^1O^1 and A^1O^1 produced The base $2r$ will then be measured from the intersection of A^1O^1 produced with the base line It is clear that the area of the trapezium below O^1 will be the same, whatever inclination is given to its side, as all will have the same base and lie between the same parallels

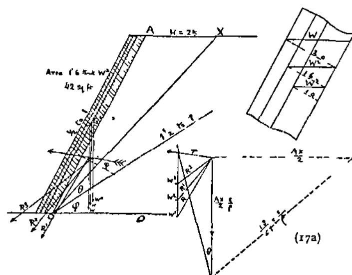
The procedure now closely follows that already described in the case of Fig 7 First the wall is divided into parts of equal height, in this case 1, 2, 3 and 4 The triangle and trapezium of earth pressure forming the corresponding divisions of the wall and of the earth pressure are next found, and vertical lines drawn through the former and horizontal lines, *i e*, parallel to their bases through the latter Through the points where these horizontal lines cut the back of the wall, the four feathered lines representing the directions of r and r^1 are drawn

(31) We are now in a position to construct the Haesslers force and ray polygon, which, owing to the varied inclinations of r, presents some difference to that illustrated already in Fig 7b First, from the nucleus O (Fig 16b), the first inclined force 1^1 is measured in its proper direction and equal in length to the half width of the triangle 1^1 on Fig 16 From its extremity, the vertical force 1, is set out equal in length to the half width of division 1 of the wall section In the same manner the other forces 2^1, 2, 3^1 3 and 4^1 4 are set out The resultants, R^1, R^2, R^3 and R^4, are clearly combinations of 1^1, 1, 1^1, 1 2^1 2 and so on, and the dotted rays drawn from the outer corners are the resultants of the odd numbers 1^1 1 2^1 1^1 1, 2^1 2 3^1, etc The drawing of the reciprocals on the wall section is identical with that already described with reference to Fig 7 and need not be repeated The intersections of R^1, R^2 R^3 and R^4, with the base lines of the divisions 1 2 3 and 4, give four points on the line of pressure The relative position of these proves the stability of the wall The weakest point is at the base, and it is evident that the stability of the wall would be considerably improved by the addition of an off set from the face near the lower end

(32) The centre of gravity of the whole wall falls without the inner toe It is not shown in Fig 16 Thus without the earth backing the wall would fall backwards This was provided for during construction by tightly packing the back of the wall with rammed earth as the work proceeded Several walls of similar light pattern were constructed by Brunel on the Great Western Railway

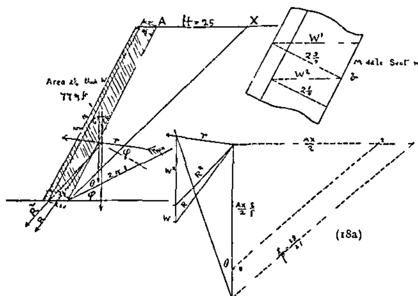
A curved face batter is very suitable for walls 30, or over 30, feet in height

as it corresponds more closely with the line of pressure and results in a more economical distribution of material For irrigation works, however, this



FIGS 17 17a

section is not very suitable except in the case of lock walls, which really belong to navigation canals



FIGS 18 18a

(33) Retaining walls, which overfall to such an extent as to be supported entirely by the earth, instead of *vice versâ*, are termed *pitched* or *riprapped* slopes Figs 17, 18 and 19 are examples in question.

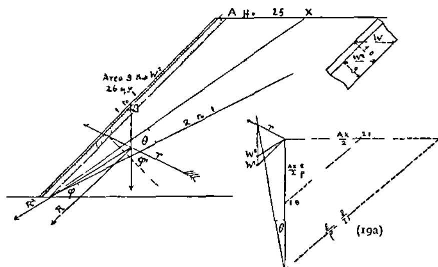
In Fig 17 the pitched slope is ½ to 1, and the diagram exhibits tentative methods of ascertaining what thickness of pitching or riprap will be

required The procedure is identical with those already described for obtaining the centre of pressure on the base of retaining walls

Trial is made of three thicknesses 2 feet $1\frac{1}{2}$ feet and 1 foot.

W in the force polygon Fig 17a has therefore three different values and consequently R has likewise three different directions and values Of the three R^2 intersects the base consequently the intermediate thickness of $1\frac{1}{2}$ feet is sufficient for purposes of stability

In Fig 18 the slope is likewise $\frac{1}{2}$ to 1 but ϕ the angle of repose, is given an inclination of 2 to 1 In this example two trial mean thicknesses of $2\frac{3}{4}$ and $2\frac{1}{2}$ feet are adopted the pitching being made 6 inches thicker at the base than at the surface Of the two thicknesses the greater or R^1 will be the correct one to adopt



Figs 19 19a

In Fig 19 the slope is 1 to 1 with the angle of repose 2 to 1, showing that a 9 inch thickness of pitching is more than sufficient

These examples prove the economy in the substitution of a masonry pitched slope for a retaining wall A river weir designed on this principle is illustrated in Fig 24

(34) Varieties of solid retaining walls used in irrigation works may be classified as below as regards section —

- A walls with horizontal crests with earth pressure to base
- B walls with sloping crests with earth pressure to base
- C, walls with earth pressure for a fraction of height above foundations
- D, walls in parallel pairs termed land wings or straight returns
- E, single land wings or straight return stop walls with earth pressure equal on either side
- F, surcharged walls
- G, walls with unequal earth pressure on both sides
- H, weir walls with earth backing to near crest which is overlaid by water

as it corresponds more closely with the line of pressure and results in a more economical distribution of material For irrigation works, however, this

FIGS 17 17a

section is not very suitable except in the case of lock walls, which really belong to navigation canals

FIGS 18 18a

(33) Retaining walls, which overfall to such an extent as to be supported entirely by the earth, instead of *vice versâ*, are termed *pitched* or *riprapped* slopes Figs 17, 18 and 19 are examples in question

In Fig 17 the pitched slope is $\frac{1}{2}$ to 1, and the diagram exhibits tentative methods of ascertaining what thickness of pitching or riprap will be

ordinarily built are not so economical as the sloping crest flank wall illustrated in Figs 20b to 23b Further in this latter disposition the flank walls form a guide for the water a function which is valuable in case of increased current as occurs below a fall

(36) Fig 20c 21c and 23c represent a type of splayed wing now in much favour for various reasons for use up stream generally in combination with (*b*) down stream

This wall is naturally more expensive than a splayed and crest sloped wall but it has the advantage of great flank protection against soakage or percolation under a head of water and in addition by causing the widening of the embankment enables the abutment to be made much shorter or if the latter is already wide carrying arches of a bridge it forms a level space close to the work and clear of the road way which space is found useful for many purposes It further guides the current into the opening which is always less than the water surface of the channel

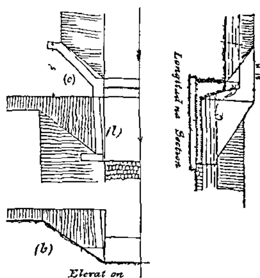


Fig 21

With regard to (*b*) in plan and elevation in Fig 20 these are views of a sloping wing with crest parallel to the axis of the work the base or spring line of the batter being splayed outward at an inclination to the crest of ratio of batter × ratio of fall of crest In this sketch the face batter being 1 in 10 and slope of crest 1½ to 1 the divergence of the base line from axis will be 1 in 15 The wing is provided with an end return 4 feet high whose length should equal its height × ratio of slope of bank *i e* $4 \times 1\frac{1}{2} = 6$ feet This return wall is surcharged but it will be strong enough in section if built of the same thickness as the end of wing abutting on to it and given a face batter

The advantage of a water wing of this description is obvious it guides the water issuing through the opening in a straight course down the channel and also protects the banks down stream from contact with water except possibly at its lower extremity which is removed far from the main work

Where as if often the case a masonry floor extends down stream beyond the abutment flank wings of this description are a necessity or else with pitched slope a dwarf wing can be substituted as shown in Fig 24a

(37) In Fig 21 a similar disposition of walls is exhibited as the *b* and *c* series in the last figure In this case the floor is not level but a drop of 4½ feet occurs in the canal bed It will be seen that the abutments are very narrow only sufficiently wide to cover the end of the cross drop wall Here

a direct return could not be adopted or if so the abutment would have to be lengthened to contain the proper top width of bank approach The efficient protection of the work from outflanking and the provision of the necessary width of bank are clearly effected in a satisfactory and economical manner by the splayed wall (*c*) (termed *flaring* wings in America)

Down stream the sloping crested water wings (*b*) display a slight difference in the way the batter is arranged The base lines are parallel with the axis of the work consequently the crest lines diverge inwards This looks some what awkward on elevation but there are cases where even a small outward splay of the water wings would be deemed objectionable it being essential that the current be guided in an absolutely axial direction

(38) Fig 22 illustrates the case of a tank fall built in a depression which has the surface of ground at about floor level In such cases the splayed

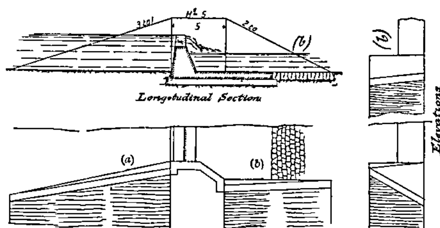


Fig 22

wings up stream are not suitable being necessarily of heavy section and the connection with the tank embankment which in this case has a flat inner slope of 3 to 1 and runs at right angles to the axis of the work is rather awkward An abutment of width equal to that of the embankment combined with splayed sloping wings as illustrated is about the best arrange ment In this case the down stream wings (*b*) are shown set back the out lining of the abutment being modified by having the corner bevelled off to prevent sharp corners and consequent eddies in the current The dog legged pattern down stream wings shown in the next figure is a still better type

The water wings (*a*) are both shown without returns running right down to the toe of the embankment on either side They are sometimes con structed splayed as far as end of down stream floor and thence continued parallel to axis or else curved as railway under bridge wings the latter arrangement coinciding with the old Madras practice The curved walls are however rightly tabooed in modern works as being conducive to pooling *i e* a destructive rotary movement in the water which has to be guarded against

(39) Fig 23 is a modification of the conditions prevailing in the last example, showing the level of natural ground as 5 feet higher, the down-stream channel being in cutting In cases such as this the splayed wings (*c*) can be employed up stream with advantage, the length of abutment being only just sufficient to contain the end of the weir wall and the splayed returns arranged so as to afford the necessary width to the embankment

The down-stream wings are of the dog-legged pattern, the abutments being also battered as well as the wings This arrangement which has

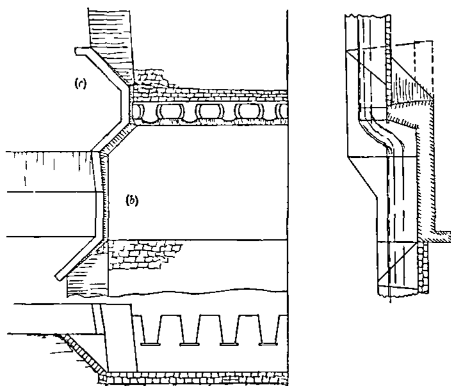


Fig 23

been adopted in the designs for canal falls given in Chap IX, is probably the best for this purpose

(40) Fig 24 contains elevation plan and section of a weir across a river which was constructed by the author in Burma This illustrates class E There being no wing walls proper, the throw back of end of embankment is effected by continuing the weir in either direction with a raised crest, the single flank walls thus answering the purpose of the double land wings In this case the flank continuations were stepped up in foundation, and, owing to the water pressure to which they are subject, were carried on of the same section as the weir proper A photograph of this work is shown in Chap IX

The banks below this weir were pitched with stone, and the floor was formed of cribbed stone planked over and further protected by a subsidiary

crib weir of loose stone, so as to form a water cushion This style of construction was necessitated by circumstances, and can hardly be deemed otherwise than a temporary arrangement, although it has lasted some years The half section marked 24a shows the probable eventual substitution of a masonry floor flanked by dwarf walls and stone pitching laid in mortar A similar design for a canal fall is exhibited in Fig 4, Chap IX

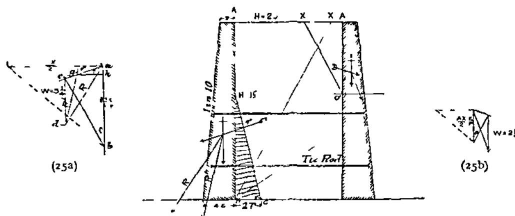
Having thus touched upon the subject of the disposition of retaining walls as are commonly used on canal works on plan and elevation, it is now

Figs 24 and 24a

proposed to continue the investigations of walls under abnormal conditions of earth pressure

(41) Fig 25 represents the section of a pair of land or direct return wings, such as are commonly employed in railway over bridges It will be seen that *OX* the plane of rupture, clears the opposite wall, and consequently the earth pressure on *AO* is identical with that which would take place were the opposite wall non existent This would apply equally to either wall, and so, notwithstanding the reduced width of the earth retained, no diminution of the section of the walls is admissible This fact causes this type of wall, if of any considerable height, to become an expensive construction, and so it is superseded where practicable, as already noted, by sloping wing walls. Owing, however, to the small space between the return wings, which seldom exceeds 20 feet, all undue earth pressure can be neutralised by the simple device of connecting the walls together with iron tie rods If this is done, a very light section of wall only need be provided, and thus equipped, the land wings can compete successfully as regards economy of section with the other types

(42) In Fig. 25 the walls are shown of reduced section, the face batter being 1 in 10, with back vertical. In Fig. 25a the usual triangles of forces *abc* and *cda* are constructed, giving the values of r and R. The reciprocal of the latter resultant transferred to the section falls well beyond the outer toe of base. In designing the section of the tie-rods required, credit should be given to the wall for the proportion of the horizontal component of R absorbed by it, the balance being provided for by the tension in the tie rods. A simple graphical process will solve the point. According to the rules adopted for the design of retaining walls, the resultant R should cut the base one-eighth of its width recessed within the toe. If this proportion be set off on the section, and a line R^1 drawn from this point to the intersection of the feathered arrow r with the vertical through the centre of gravity of the



Figs. 25, 25a, 25b.

section, R^1 will then represent the proper inclination of the modified resultant. Now on Fig. 25a draw a line dg parallel to R^1 from the extremity of W, intersecting r at the point g. The intercept cg then represents the proportion of r which is absorbed by the wall, and the remaining ga that which has to be neutralised by the tension of the tie-rods. As, however, the tension is horizontal, the horizontal equivalent of this inclined force need only be considered; gh or r^1, drawn horizontally, will then represent this force.

Reverting to Fig. 25, with a value of $H^1 = 15$ feet, the line of pressure will fall at about the required distance within the toe of that base, so that above this point there is no necessity for extraneous assistance. Below, however, the line of pressure will trend towards the face, eventually falling outside altogether, as is shown by a final resultant R. The reciprocal triangle of horizontal earth pressure, reduced to the corresponding area of masonry, will be a triangle H^1OO, having its apex at H^1, 10 feet below crest, and the width of its base being marked off $= 2r^1$ on Fig. 25a.

(43) The total horizontal stress to be carried by the ties per unit length will be the area of the triangle $H^1OO \times 62.3 \times \rho$, but as it is proposed to place the ties 5 feet apart, the stress to be provided for will be the above

expression multiplied by 5, or $\frac{3\,8}{2} \times 15 \times 2\,1 \times 0\,28 \times 5$ tons $= 8\,4$ tons The quantity 028 is the weight of a cubic foot of water expressed in fractions of a ton, as 62 3 is the weight in lbs

Two tie-rods will be required, the lower one will be placed at the centre of gravity of the pressure triangle, *i e*, 5 feet above the base of the walls This tie will neutralise the whole of the horizontal stress, viz, 8 4 tons, but another will be required at a higher level to counteract the overturning movement above the 5 feet level, as the wall might break off here, and fail by revolving on its outer toe The area of that portion of the triangle lying above the lower tie level is one half of the whole, of this one-third will be transmitted to the apex, two thirds to the base Hence the upper tie will carry one-sixth of the whole, *i e*, 1·4 tons, and the lower the remaining five-sixths or 7 tons

Taking 5 tons as the safe tensile strength of iron, the upper tie rod should have a sectional area of 3 square inches and the lower 1 4 square inches One $\frac{5}{8}$ inch diameter rod would answer for the upper and $1\frac{3}{4}$ inch for the lower, placed at 5 feet intervals

(44) The reduced area of land wings is $82\frac{1}{2}$ square feet, as against $137\frac{1}{2}$ of the normal section (*vide* Table I), and $\frac{W}{H}$ is one-thirteenth only This shows the great economy effected by the introduction of cross ties, the cost of which is trifling The ties should be provided with large plate washers, and have a head at one end and screw nut at the other The washers need not appear outside the wall, being placed in a recess near the face, which can be closed up when the final tightening has taken place

The reduction in section rendered possible by this arrangement leads to the more extended use of this description of wall, particularly in the case of masonry pitched slopes being substituted for water wings, a style of construction used for river works, but which could be well adopted for canal falls or other cross canal works

(45) In canal works in which a drop exists in the bed, it is a common practice to take the foundations of the up-stream wings down to the same level as those situated below the fall The object of this arrangement is to obviate any possibility of saturation of the bank and percolation of water through the flank of the work In Figs 21 and 23 this is shown in the splayed up-stream returns (*c*) common to both plans As canal falls are located so as to be in moderate cutting, the lower portion of the splayed walls is subjected to no earth pressure below the level of the bed of the upper reach, and reinforced concrete sheet piling could be adopted below this level instead of deepening the wall foundation

These walls, then belong to class C, *i e*, walls subjected to earth pressure for only a portion of their depth As misapprehension undoubtedly exists regarding the action of forces in such cases, as is clear from many existing examples, it will be well to devote more than usual attention to the design of walls under similar conditions

(46) Fig 26 illustrates the typical case of a vertical wall 30 feet high, the lower 10 of which is sunk in a trench in the solid ground and can

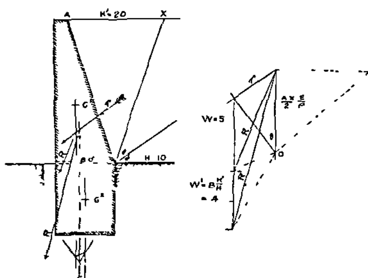


Fig 26.

consequently be safely considered as free from earth pressure below this level The base width at O is made $\cdot 4H$, in accordance with the tabular formula, and the foundations are carried down vertically The resultant of

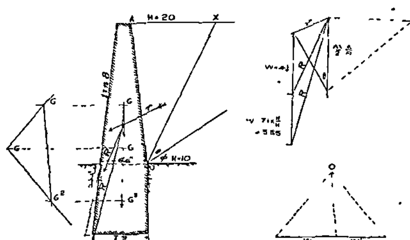


Fig 27

the upper portion R cuts its base at the proper point, but the final resultant R^1 falls without the base of the whole wall

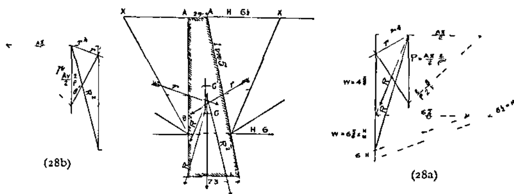
(47) Fig 27 represents exactly similar conditions, but in this instance, the wall face being battered at 1 in 8, the base width is $\cdot 4H - 1\frac{1}{2}$, and below the level H^1 the back is carried down vertically, while the face batter continues In this case R^1, the final resultant, cuts the base of the whole

wall within the outer toe, proving the stability of the section though it would be improved by still further widening the base, giving an increased batter below H^1, as is shown by the dotted line

The area of the wall in Fig 26 is 180 square feet, that in Fig 27 is 156 square feet, the former is in unstable and the lesser section in stable equilibrium If the base of Fig 26 were widened to bring R^1 within the toe at the proper place, the discrepancy of areas would be still further intensified in favour of the lighter section *

(48) As illustration of the too common neglect of this point, a few examples of actual sections will now be given and analysed

Fig 28 represents the section of a retaining wall on an existing canal fall with vertical face and sloping stepped back, which latter has been



Figs 28 28a 28b

converted for simplicity sake into an equivalent batter The wall is of brickwork and has been credited in the graphical calculation with the usual specific gravity of 1 8, that of the earth being taken at the same value It will be noticed that the final resultant R^1 falls just without the outer toe of the base The wall should therefore have a further widening of the base in front and corresponding reduction behind The front addition would have to be about double that lopped off the back, as the latter operation would tend to throw the centre of gravity forward and this tendency would be further accentuated by the addition made in front

(49) Now, supposing the wall reversed in section, *i e*, keeping to the same dimensions Let the earth pressure be on the vertical side The effect is duly worked out in Fig 28b, and R^2 is the final resultant This, it will be seen cuts the base 2 feet within the outer toe The great effect of the reversal of the section is plainly manifest Not only is the shape of the section thus reversed more suitable to sustain the thrust of the earth, but the area of the triangle of earth pressing on the wall is also much reduced The greater the back slope, the larger is also AX and r This is

* It may be noted however that the bearing capacity of foundation soil increases at the rate of about ½ ton per square foot for each foot of depth —Ed

somewhat discounted by the greater inclination of r to the horizontal when the back is inclined this inclination is the graphical allowance for the weight of earth supported on the back slope of the wall

With a vertical back p is less than with an inclined back but the great difference in the two values of r^1 and r^{1k} the horizontal component is due to the angle θ which is at a minimum with a vertical back

(50) Thus we see that a vertically backed wall is the most economical the batter being all thrown out on the front it is only surpassed when the back as in the case of Fig 15 has an inward deflection Exactly the same as will be seen later applies to dams subject to water pressure the vertical backed wall being the most economical design

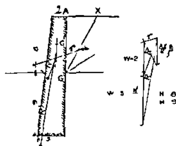
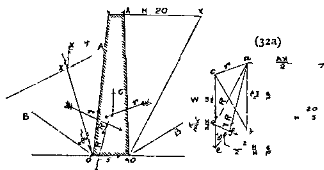


FIG 31

(53) As already observed it is a common practice peculiar to irrigation works to take the foundations of the up stream wings in case of a drop in canal bed right down to the lower level consequently walls above the drop wall or weir are not subjected to earth pressure below the level of the canal upper bed

Fig 31 represents a case in point and is a section of the splay up stream wall (c) shown in Fig 21 That figure represents a canal notch fall

(54) One further problem is required to be solved before closing the subject of the sections of retaining walls and that is the case where earth pressure exists on both sides of a wall acting in opposition viz class G The method of combining these forces with the weight of the wall is illustrated in Fig 32



FIGS 32 and 32a

The profile represents a 20 foot wall with earth pressure up to crest at the back while part of the face is covered by a falling slope This case is identical with that prevailing in connection with land wings at the ends of which the earth slope on the face reaches up to the outer crest and pivoting round on this point forms a half cone pressing on the face of either land wing thus forming in elevation a slope against the face

In Fig 32a $a\ b\ c\ d$ forms the usual combination of force triangles closing in R We have now to combine R with r^1 the inclined force acting in the opposite direction This combination can be effected on the same figure by continuing the load line cd so that de equals the relative area of earth weight

which acts on the face of the wall This will be $\frac{A^1X^1}{2} \times \frac{H^1}{H} \times \frac{\epsilon}{p}$ Then from the extremity e the θ angle is set off, intersecting the inclined line r^1 drawn through d at the point f The latter, joined with the starting point a, gives the direction and comparative value of R^1, the final resultant, i e the resultant of r^1 and R In Fig 32 R is drawn as usual from the intersection of r and G till it meets the line r^1 This last point is the starting point for the final resultant R^1, which is drawn through the base Fig 2 When the earth pressing on the wall has a horizontal terrain as is the case in many direct return wings, the procedure is the same except that O^1X^1 bisects the angle $A^1O^1B^1$

Sloping Wings on Plan and Elevation

(57) The author is strongly in favour of the adoption of inclined backs for retaining walls instead of vertical stepped backs In the former case the earth backing on settlement closes firmly against the inclined surface of the wall whereas with stepped backs the settlement is necessarily uneven and hollow spaces are certain to be formed, this has been noticed in actual cases, deep holes having been found adjoining the wall, the bank in settling having evidently cut itself clear * of the offsets of the back, leaving spaces down which the water finds its way, causing further settlement and disintegration

The advocates of stepped backs for retaining walls allege that the system possesses the following advantages firstly, of increasing the friction of the earth against the wall thus adding to its stability, and secondly of affording a more solid support to the superincumbent earth backing which can thus be credited to the wall as an increment to its weight Both these assumptions are questionable The unequal settlement which is sure to occur with stepped backs to the wall must tend to decrease instead of increase the co efficient of friction As instead of the whole of both surfaces being in close contact, they will be so only at intervals, the second consideration is not based on scientific grounds The battered back, whether in steps or in one plain surface performs identically the same statical function viz its resistance to external forces must be normal to its surface Whether this surface is composed of several horizontal and vertical portions or is in one plane its reflex action is the same as the vertical and horizontal forces if resolved are identical in direction with the force of resistance normal to the plane surface The weight of earth supported is duly credited to the wall by the normal inclination of the force r to the back which deflects the direction of this force downwards With a wholly vertical back the direction of r would be horizontal, if the friction of the two surfaces were neglected In such cases no earth rests on any part † of the wall Thus it is clear that the stepped back possesses no

* It seems reasonable to suppose that in such cases the earth has no decided tendency to press against the wall which need not offer ground for complaint The filling in of the holes is merely a matter of maintenance and repairs —Ed

† This seems to visual se the earth as overhanging the wall without touching in which case the retaining wall would seem to be superfluous except for appearances —Ed

advantages whatever over the plane batter, while it is undoubtedly more troublesome to design and to build

The sectional area of retaining walls can be largely reduced if the panel counterfort type of reinforced concrete is adopted, and economy in cost will also result provided the labour conditions and absence of local supplies of lime and material render the use of Portland cement necessary and desirable Where, however, such abnormal conditions do not obtain, the adoption of a slighter section composed of the more expensive material will not present any advantage in point either of cost or efficiency An example of R C retaining walls is given in Chap IX

Editor's Notes

(A) [illegible] relied on in th[illegible]

(1) The "wedge" theory of Coulomb, dating from the year 1780, as modified, or improved, by later investigators, and

(2) The theory of conjugate stresses in a solid under pressure, as enunciated by Rankine in the year 1852

Our author has adopted the wedge theory, as expounded by James Chalmers, C E, in the year 1880 The graphic processes employed by Chalmers are lucid and, in the main, excellent, but in one material respect they seem to be in error That is where he assigned, to the angle of friction between the earth and the back of the wall, a value equal to only half the angle of repose of the earth itself Thus, according to him, $\phi' = \frac{1}{2}\phi$, but he gave no reason for this decision

According to Boussinesq, a Belgian professor, and perhaps the most painstaking and mathematically profuse of all investigators of this subject, the precise value of ϕ' is indeterminate, but it lies between ϕ and ϕ'', where $\sin \phi'' = \frac{1}{4} \sin \phi \ (\sin \phi + \sqrt{8 + \sin^2 \phi})$ If we assume that $\phi' = \frac{1}{2}(\phi + \phi'')$ then if $\phi = 33° \ 42'$, $\phi'' = 28° \ 28'$, and $\phi' = 31° \ 5'$ The difference between ϕ and ϕ' is so small, and the process of reckoning so elaborate, that many engineers think it good enough to assume that $\phi = \phi'$

Chalmers in making $\phi' = \frac{1}{2}\phi$, instead of $\frac{1}{2}(\phi + \phi'')$, may have done so under a misapprehension of Boussinesq's complicated mathematics, but in any case he seems to have been in error, and our author, in following him, has gone the same way In Fig 1 the latter has shown the forces q and r as meeting the vertical p at the same point, but this is only achieved by locating r about half-way to the wall, instead of only one third the way up In Fig 2 also, the location of r on AO, and of q on OX, are shown at different levels, when they ought to be at the same level $= \frac{1}{3}$ height above the base line In Fig 6 and elsewhere r is located correctly, but the location and direction of force q are not indicated The fact is that in these cases q and r do not meet at the same point, on the vertical through the centre of gravity of the earth-wedge, whereas, by the Boussinesq construction, they do so meet Some who adopt the value $\phi' = \frac{1}{2}\phi$ defend it on the ground that the angle of friction of loose earth against smooth masonry is half of that of loose earth within itself But the back of a wall can usually be made so rough, or serrated, that relative motion between earth and masonry can only occur

through shearing of the earth If in any particular case it be considered necessary to make the back of the wall as smooth as its face, it should be recognised that that will require a 25 per cent increase of base thickness In the case of the wall depicted in Fig 6, with base 12 feet thick, our author's construction gives a resultant thrust that strikes the base 1 25 ($= b/10$) distant from its toe, whereas in the case of a similar wall, but with base reduced to 10 feet, the Boussinesq construction shows resultant falling on base 1 25 feet ($= b/8$) distant from its toe In the latter case Rankine s construction gives much the same result as Boussinesq s, but our author's resultant would strike the face wall 6 inches above base and would fall outside the toe

Our author s construction, therefore, which he has derived from Chalmers, and which he has made, in par (26), Table I, the basis of all his rules of design tends to make his retaining walls about 25 per cent too thick It must be noted, however, that there are still College professors of civil engineering who adopt this construction, and that base thickness depends more upon the decision as to the value of ϕ' than on any other consideration, in cases to which the wedge-theory " is applied

(B) Our author, in common with other exponents of the wedge-theory, expresses in par (57) a preference for a sloped, rather than a stepped or offsetted back to a retaining wall Sir Benjamin Baker, in the Paper quoted in par (5) did the same Rankine, however whose theory ignores earth-friction against the back of a wall, definitely preferred the back with vertical and horizontal offsets, because he took into account the weight of earth resting thereon The sloped back, of course, lends itself as much to the one theory as the offsetted back does to the other In the case of a wall with a vertical back, Rankine's process requires a wall 50 per cent thicker at base than that of Boussinesq but in such a case a difficulty in the way of the wedge theory is that the forces, lettered by our author as p, q, r respectively, do not meet at a point

To object, however, as Sir Benjamin Baker, as well as our author and others have done, to the offsetted back on the ground that it *does not encourage the earth to press against it*, seems somewhat illogical

(C) Our author does not discuss the intensities of pressure occurring on the foundation soil of retaining walls, as, for instance, in the case of the wall considered in par (14), Fig 6 though he makes a reference to it in Chap II, where it is not really relevant Most professional treatises are vague in their treatment of this part of the problem, even if they do not altogether ignore it They are apt to lavish a wealth of mathematics and theories on the stresses of the superstructure, but say little of those on the foundation soil, which is ci [illegible] Baker pointed out, 99 per cent of [illegible] due to failure of foundation soil [illegible] to be that all the currently accepted methods of computation exaggerate the magnitude of the earth-pressure and therefore, also, the thickness of wall needed to resist it In order to restrict the dimensions of walls within the bounds indicated by practice to be ample, the theorists, therefore, adopt the conventional fiction that the resultant thrust on the base of the wall may be allowed to approach up to a point whose distance from the toe is

one-eighth of the base thickness But this is scientifically false, for it is known that, in practice, in walls of the dimensions thus arrived at, the resultant thrust will not approach to anything like that degree of closeness to the toe Our author, following Chalmers, has pointed out in par (5) that the current methods of calculation ignore an important factor of stability, which, if taken into account, would bring the resultant thrust much nearer to the centre of the base of the wall Our author explains the omission of this factor on the ground that no definite value can possibly be assigned to it

In *The Engineer* of June 11th 1926 Mr F W Woods has suggested a method whereby exaggeration of the earth thrust may be avoided, and the true resultant-thrust kept within the middle third ' of the base

As Sir Benjamin Baker remarked To assume, upon theoretical grounds a lateral thrust which experiments prove to be excessive, and to compensate for this by giving no factor of safety to the wall is not a scientific mode of procedure

CHAPTER II

GRAVITY DAMS AND WEIRS

(1) DAMS of masonry or concrete can be classified into four distinct types, in the design of which different principles are involved These are

A Gravity dams and weirs
B, Arched dams and weirs
C, Arch and buttress dams and weirs
D Reinforced concrete panel box dams

In addition to these, earth rock-fill and hydraulic-fill dams will be noticed elsewhere

(2) Where water, and not earth is upheld by retaining walls its properties as a fluid and its weight, being definitely known there is less uncertainty on the subject of the actual pressure exerted

A dam may be defined to be a wall of masonry or concrete which upholds a mass of water at its rear while its face or lower side is free from the presence of water to any appreciable extent

The waste water of the reservoir formed by the dam is disposed of in another direction by means of a waste weir or bye wash, or in rare cases by means of sluices through the body of the dam

Weirs, though often confounded with dams, differ from the latter in the following points—viz , that water overflows the crest and in consequence tail water is formed below the wall These two facts modify the conditions which are applicable to dams proper and consequently weirs demand separate treatment

SECTION I —GRAVITY DAMS

(3) The pressure of water on the back of a wall at any point varies directly as the depth so that the water pressure can be represented by an equilateral triangle with its base equal to its height, the base being normal to the rear face of the wall

The unit pressure exerted on any point in the back of the wall is represented by the ordinate of the triangle of pressure drawn parallel to its base, and the whole pressure by the sum of these ordinates, *i e*, by the area of the triangle or prism

When the back of the wall is vertical this area will be $\frac{H^2}{2}$, H being the vertical depth of water and the pressure $\frac{wH^2}{2}$, w being the unit weight of water, one thirty sixth of a ton per cubic foot

When the back of the wall is inclined, the pressure will be $\frac{wH_1H}{2}$, H_1 being the inclined height of the back of the wall, which being always greater than H, the vertical depth, the latter expression is also greater.

(4) Sections of gravity dams are invariably designed on the well known principle of the "middle third." This expression signifies that the profile must be such that the resultant pressure lines, or centres of pressure, due, first, to the weight of the dam considered alone, and, secondly, to that of the water pressure in addition, must both fall at or within the middle third of the section. These two conditions of stress are usually designated as "Reservoir Empty" and "Reservoir Full."

This stipulation ensures the fulfilment of three obligatory provisos, which are: first, the absence of tension anywhere in the section, secondly, the limiting of the maximum compressive stress in any plane to a proportion not exceeding twice the mean or average stress on the same. It further usually ensures that the angle of inclination of the resultant pressures with the horizontal exceeds that of the angle of friction of the material; this is with reference to shearing stress. A limiting value to the maximum stress allowable in the masonry of the dam is also a further necessary condition that has to be observed.

(5) The theoretically correct profile of a dam subjected to water pressure under the conditions above outlined, and also of the most economical dimensions possible, is that of a right angled triangle having its back vertical and its apex at the surface level of the water. It can be proved that the proper base width of this triangle is expressed by the simple equation

$$b = \frac{H}{\sqrt{\rho}} \qquad (1)$$

in which H designates the vertical height of the triangle and the Greek letter ρ (rho) the specific gravity of the material in the dam. This profile will be termed the Elementary Profile and is shown on Fig. 1.

A base width of such proportional dimension ensures the exact incidence of the vertical resultant force W of "Reservoir Empty" and also that of the inclined resultant R of "Reservoir Full" at the inner and outer third divisions of the base respectively. The same will naturally occur on any horizontal plane.

The fore slope or hypothenuse of the Elementary Profile, will be as $1 : \sqrt{\rho}$.

The value usually assigned to ρ, the specific gravity of the material, masonry or concrete, of which the dam is composed, is $2\frac{1}{4}$ or $\frac{9}{4}$.

In many dams this is exceeded and sometimes reaches as high a figure as 2.7, but the calculations for base width are generally founded on this value of $2\frac{1}{4}$.

As $\sqrt{2\frac{1}{4}}$ is $1\frac{1}{2}$, $\frac{1}{\sqrt{\rho}}$ will be $\frac{2}{3}$, and the correct proportion of the base

will be $b = \frac{2}{3}H$, and the fore slope or batter will be as 2 horizontal to 3 vertical

(6) The weight of a cubic foot of the material with $\rho = 2\frac{1}{4}$ is expressed by $w\rho$, w being the weight of a cubic foot of water, that is $\frac{1}{36}$ ton nearly, $w\rho$ will then $= \frac{1}{16}$ ton

The system adopted throughout this work in representing pressure in tons per square foot is clearly superior to the antiquated method still adopted in many technical works of using pounds per square inch

Large denominations are more easily grasped and retained than small ones, thus the representation of the contents of a reservoir in acre-feet appeals to the mind better than gallons which run into millions The use

Fig 1

of pounds per square inch should be confined to steam or air pressures and mechanical engineering generally, for which a small denomination is clearly suitable It may be noted there that $15\frac{1}{2}$ lbs per square inch is equivalent to 1 ton per square foot

(7) In the design of a section of a dam, pier, or retaining wall, the distribution of pressure on a plane in the section and the relations that exist between maximum stress (s) and the mean stress (s_1) are matters of the utmost importance

The mean or average stress in any lamina is that which acts at its centre, and is in amount the resultant stress, the incidence of which may be at some other point, divided by the width of the lamina in question. Such width must be measured on a plane normal to the direction of the resultant force. For example, in Fig. 1 the vertical line W represents the weight of the dam acting through its centre of gravity on its base b. The intensity of the mean stress induced at the centre of b, which being horizontal is normal to W, will be $\frac{W}{b}$. The maximum stress occurs at that end of the base nearest to the incidence of W, that is at the point A, the "heel" of the base. When as in this case, the incidence of W is at the middle third point, this maximum unit stress will equal twice the mean, or $s = 2\,s_1 = 2\,\frac{W}{b}$.

This fact is due to the application of a well-known law, defining the relations between maximum and mean stress, which is expressed by the following formula

$$s = s_1\left(1 \pm \frac{6c}{b}\right) \tag{2}$$

In this c is the distance of the centre of the lamina from the centre of pressure, or in other words from the incidence of the resultant stress. When the plus sign is used the equation gives the value of the maximum unit compressive stress at the end nearest to the centre of pressure, whether it be W or R, and when the minus sign is used that of the unit stress, which occurs at the other extremity. If the result is a minus quantity it represents a tensive, not a compressive stress set up at this extremity.

If the greater stress be termed P and the less P_1, and s_1, being as we have already seen, $= \frac{W}{b}$ when the incidence of the resultant stress W is at the centre of the base, then $c = o$ and the equation becomes

$$P = \frac{W}{b}\,(1 + 0) = \frac{W}{b}$$

and

$$P_1 = \frac{W}{b}\,(1 - 0) = \frac{W}{b}$$

That is, the maximum is equal to the mean stress.

Again when the incidence of W is at the boundary of the middle third, as is the case in Fig. 1, then $c = \frac{b}{6}$ and

$$P = \frac{W}{b}\left(1 + \frac{6b}{6b}\right) = 2\,\frac{W}{b}$$

$$P_1 = \frac{W}{b}\left(1 - \frac{6b}{6b}\right) = nil$$

Thus the maximum is double the mean.* Lastly, if the incidence of W is at

* But see Editor's notes at end of this chapter. When there is tensile stress b changes to something less b' and c to c'.—ED.

the extremity of the case, $c = \frac{2}{b}$ and

$$P = \frac{W}{b}\left(1 + \frac{3b}{b}\right) = 4\frac{W}{b}$$

$$P_1 = \frac{W}{b}\left(1 - \frac{3b}{b}\right) = -2\frac{W}{b}$$

In this case the maximum unit compressive stress is four times the mean while at the further end tensile stress is set up equal to twice the mean unit stress The graphical process of obtaining the same results will be explained in par (24) later

(8) Now with regard to R or the resultant pressure "Reservoir Full", this force being always a greater quantity than W, whenever the question of the maximum permissible stress in the masonry of a dam comes under consideration, it is this force and not W that is the ruling influence

With regard to the maximum pressure induced in the masonry by the force R, this plane b is not normal to the direction of R but another plane (marked on Fig 1) b_1 is so, consequently the mean stress induced by R is not $\frac{R}{b}$, but $\frac{R}{b_1}$, the mean unit stress is therefore greater than $\frac{R}{b}$ as b is greater than b_1 and equals $\frac{R}{b \cos \theta}$ θ being the inclination of R to the vertical On the horizontal base, however, $s_1 = R \cos \theta - b$

In graphical computation it is more convenient to increase R, using b as denominator the result being identical

In the force polygon 1a, if a line be set out at right angles from the extremity of the force R, the intercept $N_1 (= R \sec \theta)$ will represent the increased value assigned to R, the mean induced unit stress on the base b will then be

$$s_1 = \frac{N_1}{b} = \frac{R}{b_1}$$

This is graphically demonstrated by the hatched areas below the profile in Fig 1

(9) Designs of parts of masonry works have often to be manipulated so as not only, as in this case, to bring the centre of pressure at the middle third point of the base, but so as to reduce the maximum unit stress to a less proportion than one of double the mean, and this can be effected by manœuvring the position of the incidence of the greater resultant R to a point as near the centre of the base as possible, with a view to equalising the value of s with that of s_1

(10) The values of the resultant and of the maximum induced unit stresses due to "Reservoir Full, or 'Reservoir Empty," are catalogued

below in terms of w and of ρ. The values when expressed in tons assumes $\rho = 2\frac{1}{4}$.

W or the weight of the triangular prism $= \frac{H^2 w \times \rho}{2} = \frac{H^2}{48}$ Tons (3)

R, the resultant (Reservoir Full) $= W \times \sqrt{\frac{\rho + 1}{\rho}} = 1.2\, W = \frac{H^2}{40}$ Tons (4)

s^a or maximum unit stress (R. E.) $= Hw\rho = \frac{H}{16}$ Tons (5)

s^b ,, ,, ,, , (R. F.) $= Hw(\rho + 1) = \frac{H}{11.1}$ Tons (6)

If the limiting permissible unit stress be designated by the Greek letter λ (lambda), the limiting height of the profile will be from (6)

$$H^\lambda = \frac{\lambda}{w(\rho + 1)} \text{ or when } \rho = 2\tfrac{1}{4} = 11.1\,\lambda \quad (7)$$

Thus if λ be 8 Tons H^λ will be 89 feet
,, ,, 10 ,, ,, 111 ,,
,, ,, 15 ,, ,, 165 ,,
,, ,, 20 ,, ,, 220 ,,
,, ,, 25 ,, ,, 275 ,,

The area of the elementary triangular profile is $\frac{H^2}{2\sqrt{\rho}}$, and that of the triangle of water pressure reduced to a masonry base is $\frac{H^2}{2\rho}$.

(11) In graphical computations the base of the triangle of water pressure is invariably made equal to $\frac{H}{\rho}$, not to H. This has the effect of reducing the pressure area from that of water, of specific gravity = unity, to the same denomination as that of the masonry of the wall, or of ρ, with the result that $w\rho$ becomes a common factor in both the masonry and water pressure areas. In the triangle of forces, the value of W and of P respectively can thus be represented by the half widths of their respective areas.

This procedure simplifies construction, as the common factors H as well as $w\rho$ are eliminated, H being also common to both triangles. In cases where H is not common and consequently cannot be eliminated, the values of W and of P will have to be represented by their areas, the common factor $w\rho$ being then the only one that can be discarded. The same procedure was observed in Chap. I. Whenever actual values in tons are required, the measured length of the resultants in the force polygon have to be multiplied by those eliminated factors, *i.e.*, by H and by $w\rho$.

(12) In actual practice a dam must be provided with a crest of definite width and not terminate in the apex of a triangle. The imposition of the crest increases the stability of the section, but throws the incidence of W

the extremity of the case, $c = \frac{2}{b}$ and

$$P = \frac{W}{b}\left(1 + \frac{3b}{b}\right) = 4\frac{W}{b}$$

$$P_1 = \frac{W}{b}\left(1 - \frac{3b}{b}\right) = -2\frac{W}{b}$$

In this case the maximum unit compressive stress is four times the mean while at the further end tensile stress is set up equal to twice the mean unit stress The graphical process of obtaining the same results will be explained in par (24) later

(8) Now with regard to R, or the resultant pressure "Reservoir Full"; this force being always a greater quantity than W, whenever the question of the maximum permissible stress in the masonry of a dam comes under consideration, it is this force and not W that is the ruling influence

With regard to the maximum pressure induced in the masonry by the force R this plane b is not normal to the direction of R, but another plane (marked on Fig 1) b_1 is so, consequently the mean stress induced by R is not $\frac{R}{b}$, but $\frac{R}{b_1}$, the mean unit stress is therefore greater than $\frac{R}{b}$ as b is greater than b_1 and equals $\frac{R}{b \cos \theta}$, θ being the inclination of R to the vertical On the horizontal base, however, $s_1 = R \cos \theta - b$

In graphical computation it is more convenient to increase R, using b as denominator the result being identical

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This is graphically demonstrated by the hatched areas below the profile in Fig 1

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(10) The values of the resultant and of the maximum induced unit stresses due to "Reservoir Full," or "Reservoir Empty," are catalogued

In addition to the above, allowance has to be made for wave action, the height of which is obtained by the following formula

$$h = 1.5\sqrt{F} + (2.5 - \sqrt[4]{F}) \qquad (10)$$

In this F is the pitch or longest line of exposure of water surface to wind, expressed in statute miles

Thus if $F = 4$ miles the extra height above maximum flood level will be

$$(1.5 \times 2) + (2.5 - 1.4) = 3 + 1.1 = 4.1 \text{ feet}$$

If $F = 10$ miles, the height works out to $5\frac{1}{2}$ feet If the height of the crest is not made higher than the allowance necessary for wave action, the apex of the triangle of water pressure should correspond with this level If on the other hand the crest is designed well clear of this point the apex will be at a lower level

(15) To revert to the pentagonal profile, if the back of a dam is not vertical but canted forward, the stability will be injuriously affected, the incidence of R falling without the middle third Thus we see that the vertically backed wall is the most economical profile for water, as it has already been shown to be for earth pressure In weirs alone it is not so

(16) We have seen that the elementary profile, or its modification the pentagonal satisfies all imposed conditions of stability up to a certain point, which is the pressure limit, below this the simple profile must be departed from and the base widened out in order that the stipulated pressure limit be not exceeded while at the same time the other conditions are maintained This widening adds immensely to the cost of a dam, consequently as high a pressure limit as is consistent with safety should be employed

Of late years the adopted values of λ, the limiting stress have largely exceeded what was previously deemed the safe limit For this bold innovation we are mainly indebted to American engineers, who have proved without doubt that pressures of 16 to 20 tons, *i e*, double the old values, are quite practicable in gravity dams, whereas in arched dams even higher pressures than these are safe

In the Roosevelt Dam (Fig 20), one of the highest in the world, the elementary profile is strictly adhered to right down to the base, a depth of 230 feet From formula (6), we obtain

$$s = \frac{H}{11.1}, \text{ i e}, \frac{230}{11.1}, \text{ or over 20 tons}$$

In contrast to this, the Assuan Dam has an imposed limit unit pressure in its piers of 5.4 tons only *

Design of Dams below the Limiting Depth H^{λ}

(17) We have seen in par (10) that the section of a dam can be carried down in accordance with the elementary triangular section or with such

* This was imposed by the designer in anticipation of a future increase in the height of the Dam —ED

modifications of it as are advisable, until the depth has a value of $\frac{\lambda}{w(\rho + 1)}$, after which the procedure becomes more complicated, the maximum stress on the masonry, which must not be exceeded, adding another factor to the problem. Not only has one line of pressure, reservoir full, viz., N, to fall at the middle third, but the base width will have to be increased in a greater ratio to satisfy the conditions of stress limit. This could be effected by means of trial and error by graphical process, but would be very troublesome and figured calculation from formulas deduced by analytical methods will be found easier to use. This difficult subject has been very ably treated in the "Principles of Water Works Engineering," and an excellent example of the method of working out the varying width is given in the above work, which will be reproduced below in a somewhat condensed form.

The solution of this problem is based on the following:—If a rigid body rest on a horizontal surface, the distance of the centre of pressure from the extremity of the base at which the maximum stress occurs is—

$$l = \frac{b}{3}\left(2 - \frac{s}{2s_1}\right) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad \text{(A)}$$

where b is the width of the base, s the maximum stress intensity in tons, the average pressure on the base being s_1, from which the following formula giving values of b is deduced:—

$$b = \frac{wH^2}{\lambda}\left(1 + \frac{w^2H^4}{4N^2}\right) \quad . \; . \; . \; . \; . \; . \quad \text{(B)}$$

λ the limiting and s the maximum stress being in this case identical. Here w = weight of water per cubic foot in tons, viz. $\frac{1}{36}$ ton, and H = depth of water. N = the vertical forces, viz., the weight of the masonry wall and of the water over the inner face where it is inclined, *i.e.*, Reservoir Full.*

From this equation b is found readily with a high degree of accuracy if N be known even approximately.

* NOTE.—Formula (A) can be written $e = \frac{b}{3}\left(2 - \frac{\lambda b}{R}\right)$ because in this case s_1 is the mean pressure on the base or $\frac{R}{b}$ not $\frac{N_1}{b}$; the same expression is $e = \frac{b}{3}\left(2 - \frac{\lambda b}{\sqrt{N^2 + P^2}}\right)$.

The base b is formed of three divisions: (1) e from the toe to the incidence of R, (2) f from the incidence of R to that of N, and (3) that from N to the heel of the base. Now $f = \frac{PH}{3N}$ obtained by taking the moments of N and of P about the incidence of R, and (3) is postulated to be $\frac{b}{3}$, whence $b = \frac{b}{3}\left(2 - \frac{\lambda b}{\sqrt{N^2 + P^2}}\right)\frac{PH}{3N} + \frac{b}{3}$. If the value of P, the horizontal water pressure, in terms of w and of H be substituted, the expression will have the form given in formula (B) par. (17).

With regard to formula (C). The vertical forces whose moments about the inner third ...

(1) $\frac{b_1}{3} - \frac{x}{2}$ (2) $\frac{b_1 - b}{3} - x$ (3) $\frac{3b - b_1 + 6x}{12}$. By separately estimating the weights 1, 2 and 3 the equation is very much simplified, being reduced to a simple equation, and x is found with ease. The one condition is that the incidence of N^1 must be exactly at the inner third point of the new base. This proviso insures that both H, reservoir empty, and R fall within the middle third. The length given to b_1 also insures that $s = \lambda$ the limit stress.

Having found b, or b_1, the next step is to ascertain how much of it must be under the inner face of the dam, *i e*, within the vertical through the crest, in order to bring the incidence of the resultant weight of the whole superincumbent mass of the masonry and water to a distance of $\frac{b}{3}$ from the inner toe This is arrived at by equating the moments of the vertical forces about O_1 the point in question (*vide* Fig 2), to zero, by which means we obtain the following equation, the solution of which gives the value of the distance x_1, *i e*, the projection of the base at heel of wall —

Fig 2

$$\left.\begin{array}{l}\frac{\rho u d}{24}\left(3b^2 - b_1^2 + 6x_1(b + b_1) + 2bb_1\right) - \frac{u x_1}{12}(H + H_1) \times \\ (2b_1 - 3x_1) - N\left(\frac{b_1 - b}{3} - x_1\right) = O\end{array}\right\} \quad . \quad (C)$$

H is depth of water at the base b, and $H_1 = H + d$, d being depth of lower lamina

This formidable-looking quadratic equation is not so difficult to work out, as one would judge from its inordinate length

From formula (B) the value of b_1 is obtained *i e*, the bottom width of the first strip to be added below the base, and from (C) x_1 is found, which fixes the position of the new lamina with regard to the base above

(18) The following illustration of the practical working out of the formulas (B) and (C) is given below

Fig 3

Fig 3 represents the whole section of the dam, part of which is shown in *Fig 2, the value of ρ being $2\frac{1}{4}$ and λ 10 tons, whence H, the limiting depth,* will be equal to 111 feet, par. (10).

The base width at this depth will be equal to $x + 74\ 0 = 74\ 6$, or excluding the back strip 74 0 The face line corresponds with that in the elementary triangular profile no deductions having been made

Now the weight of the section, including that of the water overlaying the portion EF of the back, is nearly 265 tons The successive values of H to be considered will be —

From crest to base of low dam	-	H	111 feet
,, ,,	1st lamina -	H_1	120 ,,
,, ,,	2nd ,, -	H_2	130 ,,
,, ,,	3rd ,, -	H_3	140 ,,
,, ,,	4th ,, -	H_4	150 ,,

The respective values of b and x (Fig 13) may be conveniently designated by b b_1 b_2, and x x_1 and x_2, also the total weight of the dam and superincumbent water may be by N, N_1 and N_2

First Lamina —At the base of the low dam (Figs 2 and 3)

$$N = 265 \text{ tons } H = 111 \text{ feet}, b = 74\ 6 \text{ feet } x = 0$$

If the profiles above b be produced down to the level $H_1 = 120$ feet, *i e*, 9 feet deeper the weight of the trapezoid of masonry thus added is

$$\frac{9 \times 74\ 6 \left(1 + \frac{120}{111}\right)}{2 \times 16}, \text{ say, equal to 44 tons}$$

Then, as a first approximation,

$$N_1 = 265 + 44 = 309 \text{ tons}$$

By equation (B), par 17, introducing the proper value of λ, or s of 10 tons

$$b_1 = \sqrt{\frac{(120)^3}{360}\left(1 + \frac{(120)^4}{5184 \times (309)^2}\right)}$$
$$= 82\ 5 \text{ feet}$$

The next step is to find x_1 corresponding with this value of b_1

By equation (C), we have

$$\frac{9}{16 \times 24}[3 \times 5565 - 6806 + 6\,(157\ 1)\,x_1 + 2 \times 6154]$$

$$- \frac{x_1}{36 \times 12}[231\,(165 - 3x_1)] - 265\,(2\ 63 - x_1) = 0,$$

that is $$1\ 6x_1^2 + 199x_1 - 177 = 0,$$

therefore $$x_1 = \frac{-199 + \sqrt{39\ 601 + 1133}}{3\ 2},$$

$$= \frac{3}{3\ 2} = 0\ 9 \text{ foot}$$

The values of b_1 and x_1 thus found, enable us to obtain a closer approximation to the weight of the trapezoid under consideration, and to determine the weight of water overlying its inner face The sum of these two quantities may be found to be 47 tons

Then a second approximation gives

$$N_1 = 265 + 47 = 312 \text{ tons}$$

Introducing this value of N into equation (B), par (17), we find, as a second approximation,

$$b_1 = 82\ 3 \text{ feet},$$

which value of b_1 is so nearly that previously used in applying equation (3) as not to affect the already found value $x_1 = 0\ 9$ foot

Thus we have at the base of the first lamina below the low dam—

$$N_1 = 312 \text{ tons}, H_1 = 120 \text{ feet}, b_1 = 82{\cdot}3 \text{ feet}, x_1 = 0\ 9 \text{ foot}$$

(19) *Second Lamina*—Proceeding as before, produce the profiles immediately above b_1 down to the level $H_2 = 130$ feet, the weight of the trapezoid of masonry thus added, together with that of the water overlying its inner face, is

$$\frac{10}{16}\left[82\ 3 + \frac{1}{2}\ \frac{10}{9}(82\ 3 - 74\ 6)\right] + \frac{120 + 130}{2 \times 36} \times 1\ 0 = 58 \text{ tons},$$

Then, as a first approximation

$$N_2 = 312 + 58 = 370 \text{ tons}.$$

Hence
$$b_2 = \sqrt{\frac{(130)^3}{360}\left(1 + \frac{(130)^4}{5184\ (370)^2}\right)}$$
$$= 92\ 5 \text{ feet}.$$

Applying equation (C), par (17), and introducing these values of b_2 and N_2, we find

$$x_2 = 1{\cdot}4 \text{ feet}.$$

Correcting the weight of the trapezoid and its superincumbent water for this value of x_2, we have as a second approximation

$$N_2 = 312 + 59 = 371 \text{ tons},$$

whence a re-application of equation (B), par (17), gives the corrected value

$$b_2 = 92\ 4 \text{ feet}.$$

Thus, at the base of the second lamina,

$$N_2 = 371 \text{ tons}, H_2 = 130 \text{ feet}, b_2 = 92\ 4 \text{ feet}, x_2 = 1{\cdot}4 \text{ feet}.$$

(20) *Third Lamina*—Produce the profiles above b_2 down to the level $H_3 = 140$ feet

As a first approximation

$$N_3 = 371 + 67 = 438 \text{ tons}.$$

$$b_3 = \sqrt{\frac{(140)^3}{360}\ \frac{(140)^4}{(1 - 5184\ (438)^2)}}$$
$$= 102\ 8 \text{ feet}$$

By equation (C), par (17),

$$x_3 = 1\ 4 \text{ feet}$$

This introduces no sensible correction in the value of N_3, and we therefore have, without further calculation, at the base of the third lamina

$$N_3 = 438 \text{ tons}, H_3 = 140 \text{ feet}, b_3 = 102·8 \text{ feet}, x_3 = 1·4 \text{ feet}$$

Fourth Lamina.—Produce the profiles above b_3 down to the level $H_4 = 150$ feet

As a first approximation,

$$N_4 = 438 + 75 = 513 \text{ tons}$$

$$b_4 = \sqrt{\frac{(150)^3}{360}\left(1 + \frac{(150)^4}{5184\ (513)^2}\right)}$$

$$= 113·3 \text{ feet}$$

By equation (C), par (17),

$$x_4 = 1·4 \text{ feet.}$$

Again no sensible correction is introduced into N_4, and we have at the base of the fourth lamina

$$N_4 = 513 \text{ tons}, H_4 = 150 \text{ feet}, b_4 = 113·3 \text{ feet}, x_4 = 1·4 \text{ feet.}$$

Summarising these particulars, we have :—

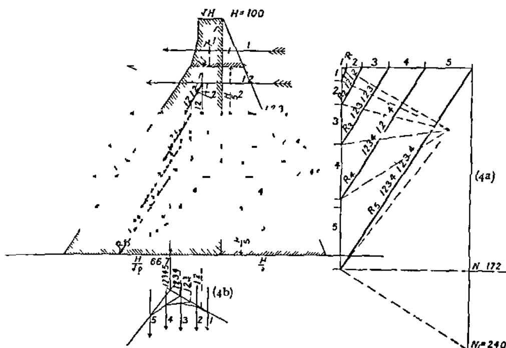
Depth below crest of dam (H)	Breadth of base (b)	Weight of masonry	Breadth measuring from axis under inner face (x)	Weight of water over inner face
Feet	Feet	Tons	Feet.	Tons
0	10	—	—	—
111	74·6	264	0·6	⅔
120	82·3	308	1·5	4
130	92·4	362	2·9	9
140	102·8	424	4·3	14
150	113·3	493	5·7	20

(21) The foundations of dams are often carried for a great depth below the surface of the ground, and if a square concrete base with vertical sides is provided, closely adhering to the original cutting face or soil or rock, this portion may be considered as free from water pressure

As a general rule, however, the dam is designed for full water pressure down to its actual base Below this base a comparatively narrow trench filled with concrete and puddle has often to be excavated to great depths This is simply a curtain wall to stop any chance of percolation, and is not designed to withstand water pressure

(22) An example of the pentagonal profile of a dam 100 feet high, designed in accordance with the rules already given, is exhibited in Fig 4 The base is made $\frac{H}{\sqrt{\rho}}$ or $\frac{2}{3} \times 100 = 66·7$ feet. The crest is $\sqrt{H}$, or $·15b = 10$ feet. The base of the water pressure triangle is made $\frac{H}{\rho}$ or $\frac{4}{9} \times 100 = 44·4$ feet for reasons previously explained

The procedure of drawing the line of pressure in Fig 4 is that usually adopted for showing the points of pressure reservoir empty and full. The system consists of dividing the profile and triangle of water pressure into a certain number of equal laminas in this case 5 numbered accordingly 1 to 5, while the corresponding divisions of the triangle of pressure are numbered 1^1 to 5^1. This latter having a horizontal base all half-widths will be measured parallel to it. A series of independent combinations are now formed in the force triangles in Fig 4a viz of 1^1 and 1, 1^1 2^1 with 1 2, 1^1 2^1, 3^1 with 1, 2 3 and 1^1 2^1 3^1 4^1 with 1 2, 3 4. The centres of pressure of these three last combinations are discovered by use of the funicular 4b, and the



Figs 4, 4a and 4b

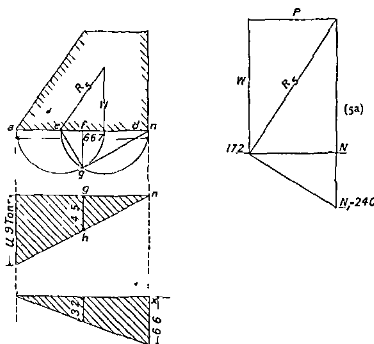
intersection of the projected vertical resultants with the horizontal and inclined forces of each combination. The intersection of those same verticals with their respective base lines give points on the line of pressure, reservoir empty. Thus each pair of forces is independently dealt with, this arrangement is so far advantageous in that error is not perpetuated. The system, however, can only be used when the back is in one straight line.

(23) In the force polygon Fig 4a R_5 is the final resultant. As explained in par (8), by setting out a line at right angles from its lower extremity, the intercept cut off from a vertical line drawn from the upper extremity gives the value N_1, which is the actual maximum resultant pressure on the base. Another intercept N is formed by a horizontal line drawn from the extremity of the load line W. This N is the vertical component, Reservoir Full.

In this case the water pressure forces being all horizontal, $N = W$,

which otherwise would not be the case In order to obtain the actual values of N and of N_1, the measured lengths in feet will have to be multiplied up by the eliminated common factors, *i e*, by $\frac{H}{5}$ or by 20 and by $w\rho$ or by $\frac{1}{16}$ ton Now N, or W measures 172 feet and its actual value will be $172 \times 20 \times \frac{1}{16} = 215$ tons This is the vertical weight per foot run of the dam N_1 measures 240 feet and similarly its actual value will be $240 \times 20 \times \frac{1}{16} = 300$ tons

(24) The graphical method of ascertaining the distribution of pressure on the base of the masonry wall, which has already been dealt with analytically



Figs 5 and 5a

in par (7) is exhibited in Figs 5 and 5a, which are a reproduction of part of Figs 4 and 4a The procedure is as follows —Two semi-circles are struck on the base with their centres at the *two-third division points*, and with radius $\frac{b}{3}$ From *e*, the point of incidence of R_5, the line *eg* is drawn to *g*, the intersection of the semi circle Again from the point *g*, a line *gn* is set off at right angles to *eg*, cutting the base or its continuation at the point *n* This point *n* is the antipole of *e*, or the neutral point at which pressure is *nil* in either sense

Below the profile another projection of the base is made From *g* a perpendicular is let fall cutting the new base line in g^1, and its continuation g^1h is made equal to the mean unit pressure or to $\frac{N_1}{b}$ Another perpen-

dicular is then let fall from n, cutting the new base in h_1 and then the points n^1 and h are joined and the line continued till it meets the perpendicular marked n from the toe of the base Where n does not coincide with d the heel of the base a further perpendicular must be drawn from the heel The hatched trapezoid thus enclosed represents the distribution of pressure In similar manner the pressure due to W or N the vertical load, is shown

(25) We have already seen (par (23)), that the value of N_1 in Fig 4a is 300 tons The mean pressure will then be $\frac{N_1}{b}$ (par (7)) or $\frac{300}{66\ 7} = 4\ 5$ tons This is marked off from g^1 to h and n measures double the mean, or 9 tons

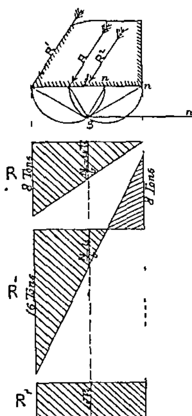
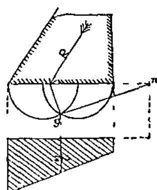


Fig 6


Fig -

In this diagram n happens to correspond with d because of the incidence of R_5 at the outer third division point

In the hatched area the intensity of pressure at any point in the base is measured by the vertical ordinate of the triangle

A similar pressure area for the load N is given, here $N = 215$ tons, and $\frac{N}{b} = \frac{215}{66\ 7} = 3\ 2$ tons, the maximum at the heel being $= 6\ 4$ tons

(26) In Fig 6 the distribution of pressure on the base due to the incidence of R, first at the toe, secondly at the two third point and thirdly at the centre

is illustrated In the first case (R^1) it will be seen that the neutral point n falls at the first third point Thus two-thirds of the base is in compression and one third in tension, the maximum in either case being clearly proportionate to the relative distance of the neutral point from the toe and heel of the base, the compression at the toe being four times, while the tension at the heel is twice the mean pressure or $\frac{N_1}{b}$ assumed at 4 tons *

In the second case R intersects at the two third point, and the consequent position of n is exactly at the heel The whole base is then in compression, and the maximum double the mean

In the third case (R^2) the line gn will be at right angles to fg, which is vertical, and consequently is horizontal, and the position of n is indefinite, the area of pressure thus becomes a rectangle with a uniform pressure of $\frac{N_1}{b}$

In Fig 7 an intermediate case is exemplified

(27) In order to illustrate Haessler's polygon as used with a curved back Fig 8 is produced

The profile is similar to Fig 4, but with a projection at the rear, the back of laminas 4 and 5 being battered outwards The face line is also recessed within the elementary hypothenuse The section is not put forward as a model but simply by way of illustration of the graphical system employed with a curvilinear back

Two force polygons, Figs 8a and 8b, are given, illustrative of two methods of graphical construction, which have identical results The corresponding force lines in both figures are parallel and of the same lengths Either can be used The small detached triangle adjoining Fig 8a is merely an enlargement of the first triangle of forces 1, 1^1, R_1, which is on too small a scale for accurate transmission to reciprocal lines on the profile

The process having been already described in par (30), Chap I, need not be repeated It differs from that shown in Fig 4 in that the forces are not grouped in sets, each pair, with their resultant, being independent of the remainder, but the whole system forms a combination of all the several parts The resultant lines of water pressure belong only to each lamina separately, and are drawn normal to the back of the wall In the former case the water pressure lines belong to a group of several laminas, and consequently if the back of the wall is not in one plane any direction given will be erroneous

(28) As the laminas of the water pressure areas have inclined not horizontal bases, their mean widths multiplied by their vertical depth, i e, by $\frac{H}{5}$, do not represent their areas, consequently $\frac{H}{5}$ not being a common factor, cannot be eliminated, all the forces then would have to be [illegible]nted by areas not half widths

* But see Editor's Notes at end of this Chapter —Ed

This can be avoided by the following simple device which will be found frequently used in subsequent diagrams —

In Fig 8 let the back of the lowest lamina *acdb* be produced upwards to

FIGS 8 8a 8b and 8c

A and *Ad* joined *Acd* is then the triangle of water pressure on the supposition that the back of the wall had the same inclination throughout as *ac* because *cd* has originally been made $= \frac{H}{\rho}$ the trapezoid *acdb* also represents the water pressure on the plane *ac*

Draw *de* parallel to *ac* and join *Ae* Then the triangle *acd* and *Ace* are equal being on the same base *Ac* and between the same parallels But the area of the triangle *Ace* is equal to $ce \times H$ consequently also the horizontally based trapezoid *acef* which has $\frac{H}{5}$ for its vertical depth is equal to the original inclined area *acdb* i e it represents the water pressure acting on *ac* The same procedure is followed with regard to the upper lamina

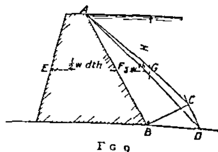


Fig 9

With this alteration which takes longer to describe than to effect the horizontal half widths of the two newly formed pressure areas will truly represent their areas as their depth $\frac{H}{5}$ is a factor common to all others

In Fig 9 a clearer illustration of the working is afforded the horizontally-based triangle ABD being substituted for its equal ABC the half width FG will then represent the area of either when H is eliminated

(29) Two further examples of dam sections are given in Figs 10 and 11 In both of these the back of the wall is given a slight batter to compensate

FIG 10

for the imposition of the crest The latter face also is not carried down vertically but is joined to where a height of $\frac{3}{4}H$ intersects the hypothenuse of the elementary triangle This thick necked profile is preferred by some authorities as affording resistance to the great temperature stresses to which the upper quarter of a dam is subject as well as to the pressure of ice

In Fig 10 the two lines of pressure are drawn through the profile Haessler s system being employed

In Fig 11 the dotted lines show the graphical reduction of the actual half widths which are reduced from a horizontal line to a smaller scale on the load line The final resultants of R and W on the base are alone shown The drawing of pressure lines at end is really only necessary below the limiting depth but has been done in Fig 10 and other cases in order to illustrate the graphical methods employed

(30) Some examples of actual sections confined to strictly modern types will now be given

Fig 12 is of the Chartrain Dam erected some fifteen years ago This dam may be considered as the latest exposition of French practice of which

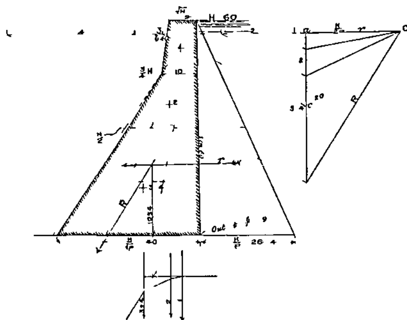


FIG 11

the Furens Dam was the first to be designed on strictly correct scientific principles The crest is remarkably light and yet stiff and the roadway

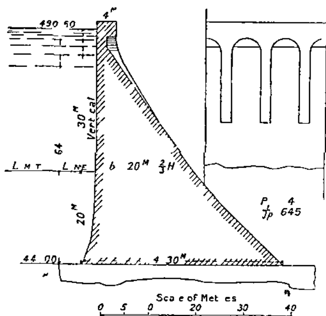


FIG 12 —Chartrain Dam

partly carried on arches overhanging the fore slope which arrangement greatly adds to the architectural effect of the elevation

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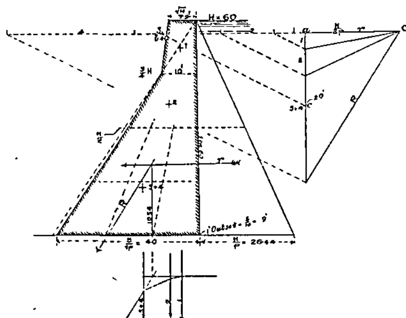


FIG. 11.

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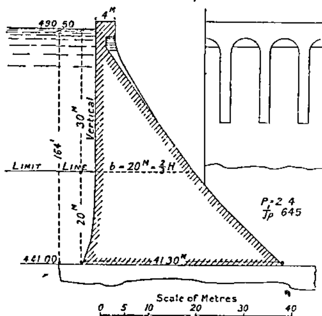


FIG 12 —Chartrian Dam

partly carried on arches overhanging the fore slope, which arrangement greatly adds to the architectural effect of the elevation.

The limit pressure is 9 1 tons, and the value of ρ is 2 4 The limiting depth is marked on the profile The back is vertical up to this point, beyond which it widens out in accordance with the principles and method adopted in this work The height H governing the base in the formula $\left(b = \frac{H}{\sqrt{\rho}}\right)$ is taken half way between full reservoir level and the actual weir crest The dam is built to a curve of 1 300 feet radius, and is over a tributary of the Loire

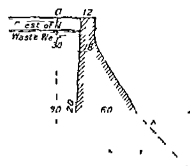


Fig 13 —Periyar Dam

(31) Fig 13 is the section of the Periyar Dam in the Madras Presidency The Periyar river is fed by the heavy rainfall of the western Ghauts By means of the dam the natural outlet of the river is blocked and the supply is thus diverted by a tunnel cut through the watershed of the Peninsula into the comparatively rainless tracts

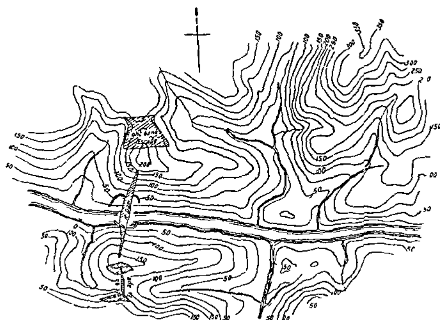


Fig 13a —Periyar Dam Site plan

east of the mountain barrier It debouches into the channel of the Vaigai river a stream running towards the east coast which will form the supply to a large canal and tank system

Surplus water is passed over a waste weir on the left in addition to which there is a wide bye wash or waste way cut in the solid rock on the right k of the dam The position of the work is shown in the site plan of

Fig 13a excepting that of the tunnel This work was constructed under great difficulties due to the unhealthiness and remoteness of the site and also due to the timidity of the Irrigation Department in not sanctioning the construction of a sluice through the body of the dam so as to allow drainage to pass during the progress of the work which greatly increased the cost and the difficulties to be overcome It can impound 156 500 acre feet and was first opened for irrigation in October 1895

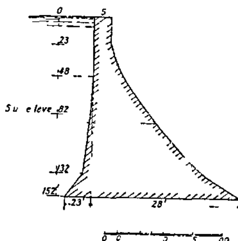


Fig 14—Marikanave Dam

(32) Fig 14 is a presentment of the section of the Marikanave Dam situated in the native State of Mysore

It has the marked peculiarity of having its escape for surplus water below full requirement the dam having been constructed so high above the ordinary working capacity of the reservoir as to enable half the highest floods to be absorbed therein It was found cheaper to do this than to provide an escape of the full flood capacity

In this dam the value of ρ is 2 4 that of λ is 8 tons It was built of rubble masonry set in Kunkur lime mortar an excellent native nodular limestone possessing hydraulic property which is found over large areas in gravel quarries composed of nothing but this material Kunkur not only supplies all the lime but is used for the metalling of roads and other similar functions

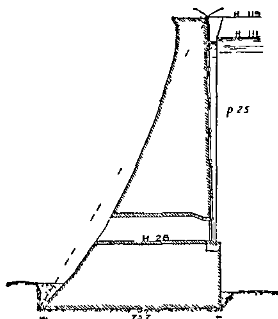


Fig 15—Bhatgarh Dam

In this dam the extreme water level is estimated at 142 feet above bed level leaving a weather board of 10 feet The elementary profile drawn on the section has its apex at the crest level as this outline appears to be more in consonance with the actual profile This work is 1 200 feet in length It is designed to store 183 500 acre-feet for irrigation of 30 000 acres of rice but it has three and a half times that capacity between sluice floor and F S L and five times the same up to maximum flood level

(33) Fig 15 is a section of the Bhatgarh Dam, which forms part of the Nira Left Canal Head works in the Bombay Presidency Its height is 119 feet, and it has the appearance of having been designed according to the water level at RL 111 the divergence of its face from the hypothenuse of the elementary triangle with apex at 119 0 being too great It is provided with fifteen low-level vents, each 4 feet wide by 8 feet deep, and also turbine sluices

This dam is 3,020 feet long and 127 feet high above lowest point in foundations The prescribed limiting pressure, the vertical component only being considered is 8 tons per square foot, and the average weight of the structure is stipulated not to be less than 160 lbs per square foot Although the greater part is composed of concrete yet this limit was actually exceeded,

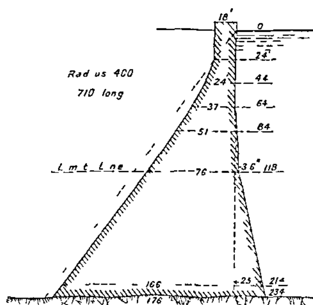


Fig 16 —Cheeseman Dam Colorado

some of the stone put in weighing as much as 190 lbs per cubic foot The concrete was mixed with large blocks of loose stone, which were inserted in the work as it proceeded These saved mortar and added to the weight and also efficiency of the concrete mass The sides were lined with ashlar, and the lower portion was built with rubble masonry The sluice ways were lined with cut granite ashlar blocks On the section, the elementary triangular profile has been drawn, the base $\frac{H}{\sqrt{\rho}}$ being 71 feet, ρ having a value of over $2\frac{1}{2}$ The actual base is 74 feet, probably to allow for compensation for the ventage

The fifteen sluices are spaced 17 feet apart, so they do not require any special precautions of limit of pressure with regard to the piers such as are necessary in the Assuan Dam It impounds 122 000 acre-feet *

* This dam has now been superseded by a new dam 160 feet high which has been built 200 feet downstream of the old one The new dam was begun in the year 1912 and its com

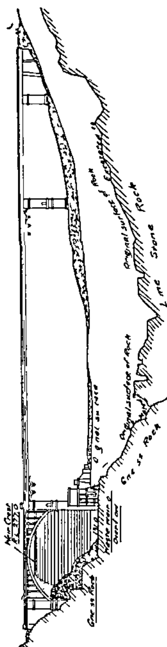


FIG 17

FIG 17a

FIGS 17 and 17a

(34) Of gravity dams the Cheesemn Dam in Fig 16 is one of the highest existing examples It is built on a curvature of 400 feet radius, and is for the water supply of Denver City, Colorado The spillway for surplus flood

pletion in 1928 is anticipated. It will have a storage capacity of 551 000 acre-feet and will supply water to the new Nira Right Bank Canal designed to irrigate 114 000 acres annually — ED

water is a bye-wash constructed in a narrow gap in the granite ridge, and is 300 feet wide

The hypothenuse of the elementary profile is shown dotted in the section till it meets the actual base At this level the value of *s* works out to 17 4 tons This graphical calculation is not shown on the figure

Calculated as an arch from the " Long " formula given in Chap III, the pressure comes to 18 tons

(35) The new Croton Dam for the water supply of New York City is a recently constructed work of great magnitude The works consist of a dam carrying a roadway 20 feet wide on the crest and at right angles to it the waste weir is set, forming one side of the reservoir By this remarkable arrangement one flank of the river valley is utilised for the weir and the

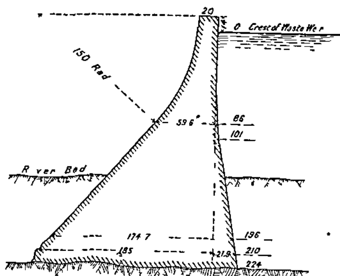


FIG 17b—New Croton Dam N Y

escape channel which is walled off from the actual river channel below the weir The original waterway down stream is filled up to a high level and utilised as a park or recreation ground

The dam portion of the work is 1,168 feet and the weir 1 000 feet in length The plan and elevation of the work, for which the author is indebted to the *Engineer*, are given in Figs 17 and 17a, the section of the dam in Fig 17b The section of the dam is remarkable for its great base thickness and the fact that the greater portion of it is buried out of sight below the bed of the reservoir The maximum unit pressure is 11 tons Owing to a fault in the rock foundation, it was deemed essential to carry the solid masonry down to this immense depth through 150 feet of sand and gravel The rock eventually met with formed probably the bed of the ancient glacial river and the material dug out was deposited in that remote age

(36) The Cross River Dam (Fig 18) is another New York work, but on different lines to the two previous dams, having a closer correspondence with

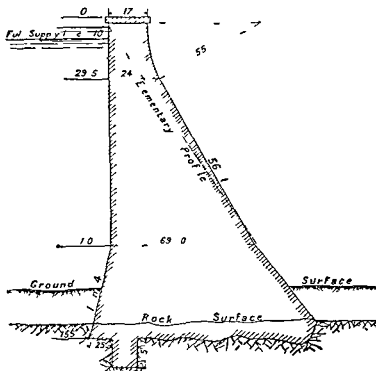


FIG 18 —Cross R ver Dam N Y

the elementary profile The limiting stress is apparently 10 tons only The neck of this dam is designed thicker than in the last two examples on account of ice

(37) Fig 19 is a section of the Ashokan Dam for the New York Water works extension The section is a very thick one extending in the upper

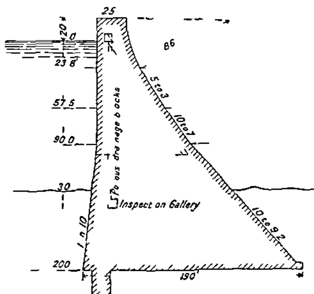


FIG 19 —Ashokan Dam N Y

part 5 feet beyond the hypothenuse of the elementary profile, with the apex of the latter placed at crest level, if the apex were placed lower at half-way up the weatherboard which is no less than 20 feet deep, the discrepancy would amount to 10 feet Both lines have been drawn on This dam is fitted with a vertical line of porous blocks connected with two inspection galleries This is in accordance with recent German practice to enable any leakage which would induce hydrostatic pressure to be stopped and drained off It is probable that the introduction of this refinement is the cause of the unusual widening of the section The dam is constructed of cyclopean masonry set in P C concrete With the great interests involved, it was probably the determination of the designer to be absolutely on the safe side

(38) The Roosevelt Dam is given in Fig 20 It was completed in the year 1911 It is built on a curve of 410 feet radius to centre of crest, and

Fig 20—Roosevelt Dam

416 feet to extrados of the arch It has been, however, designed as a gravity, not as an arched dam the curve being introduced for greater security, as it allows of freer movement under changes of temperature than the straight alignment However that may be if the apex of the elementary triangle were situated as is probably the actual case at 10 feet above the floor of the bye wash or waste way the value of H will be 230 feet requiring a base width of 154 feet according to formula 6 which is close to what it actually scales The maximum unit stress has been worked out and amounts to

16 tons If the stress were calculated as if the dam were an arch, it would work out by the long formula No 3 given in the next chapter, to 20 8 tons,

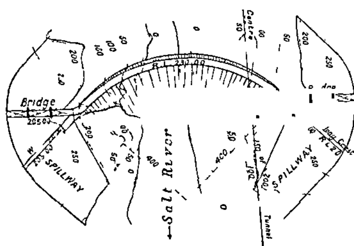


Fig 20a —Roosevelt Dam

so that the stresses are not identical whichever view is taken of the status of the dam The profile is free from the faults of being too thin or too thick in the neck as is apparent in the last four examples and is considered

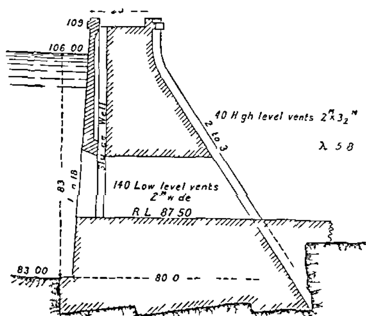


Fig 21 —Assuan Dam

unexceptionable in every respect As noted above the section adheres closely to the elementary profile and forms a powerful advocacy in favour of the simple style of pentagonal profile favoured in this work

The site plan given in Fig 20a forms an instructive example of the

arrangement of spillways cut in the solid rock out of the shoulders of the canyon, the material thus obtained being used in the dam itself These spillways are each 200 feet wide, and are excavated to 5 feet below the crests of the waste weirs which cross them This allows of a much greater discharge passing under a given head than could be the case were a simple

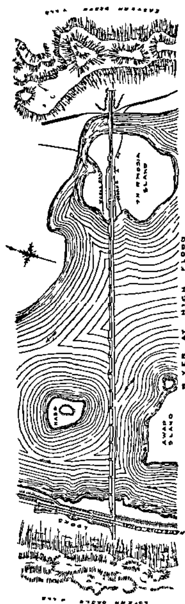
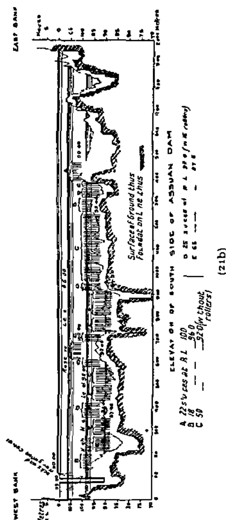


Fig 21a —Assuan Dam (crest at 109 0)



(21b)

bye wash provided without any drop wall The afflux is consequently diminished

This dam will impound the enormous quantity of $1\frac{1}{4}$ million acre-feet It spans the Salt River in Arizona, and is a U S A reclamation project On the same river below this work is situated the Granite Reef Weir, of which mention is made in the next section.

(39) Fig 21 is a section of the Assuan Dam at its deepest part This great work is built across the Nile at the first cataract Its height is only 85 feet but like most Eastern works it makes up in length what it lacks in height which former is 6 400 feet

The area now impounded is 863 000 acre feet but it is intended to raise



Fig 22 —Assuan Dam Egypt

the crest to R L 116 0 *i e* by 7 metres This will double the capacity of the reservoir which will exceed that of the Roosevelt Dam Even then its height will be only 106 feet one third of that of the Shoshone arched dam This work is principally remarkable as being the only solid dam which passes the discharge of a large river like the Nile through its body for which purpose it is provided with 110 low level sluices each 23 feet

deep by 6^1 feet wide and forty high level sluices 110 feet high and 6 feet wide They are capable of discharging 500 000 second feet the velocity in the low level sluices being as high as 20 feet per second As a matter of fact however *the rise and the fall of the Nile being very regular* the level in the reservoir can be so adjusted as never to strain the sluices to anything like this extent

The profile is by no means a bold one the unit pressure in the piers being limited to 5 8 tons only

Even when the dam is raised the unit stress in the piers would not exceed 10 tons Nevertheless it is intended to widen the whole base in order to maintain the restriction in unit pressure which is now in force To effect this an outer skin will be built on the down stream side and the space between it and the present face of the dam will be filled up with cement concrete *

Other sections of this dam showing details of the Stoney sluice gates and frames are given in Chap XIV Figs 14 15 16

The photograph Fig 22 shows some of the sluices in operation and gives a good idea of the extent of the work

Section II —Gravity Weirs

(40) When water overflows the crest of a dam it is termed a weir and some modification in the design of the section generally becomes necessary and not only that but the kinetic effect of the falling water has to be provided for by the construction of an apron or floor which in many cases forms by far the most important part of the general design This is so pronounced in the case of dwarf diversion weirs over wide sandy river beds that the weir wall itself forms but an insignificant part of the whole section and a separate chapter will have to be devoted to the elucidation of the complicated problem presented for solution At present the section of the weir wall alone will be dealt with

(41) Fig 23 is a typical section of a trapezoidal weir wall with water passing over the crest The height of the crest of the wall as before is designated by H that of the depth above crest to reservoir level by d The total height to the upper water level will therefore be $H + d$ The triangle water pressure will have its apex at the surface and its base will be $\left(\frac{H+d}{\rho}\right)$ or $\frac{H}{\rho}$ as previously but it will be truncated at the crest level and the actual *water pressure acting on the wall is the lower part of the triangle a trapezoid* whose base is $\frac{H+d}{\rho}$ and whose top width is $\frac{d}{\rho}$ Its area will therefore be if the back of the weir is vertical

$$A = \left(\frac{H+d}{\rho} + \frac{d}{\rho}\right)\frac{H}{2} - H\left(\frac{H+2d}{2\rho}\right) \qquad (11)$$

* This work was carried out in the year 1911 —Ed

If the back is inclined, the area will be

$$A = \left(\frac{H + 2d}{2\rho}\right) \times H' \qquad . \qquad . \qquad (11a)$$

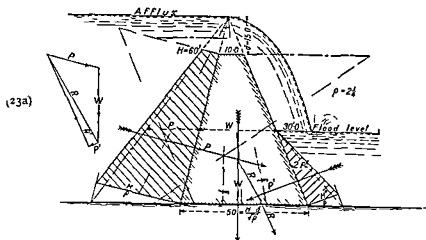


Fig 23

H' being the inclined length of the back of the weir wall The distance of its point of application above the base

$$= \frac{H}{3}\left(\frac{H + 3d}{H + 2d}\right) \quad . \ . \ . \ . \ . \ . \ . \ . \ . \quad (12)$$

(42) With regard to the weir wall, the elementary triangular profile, so useful in the case of dams, will be found to be equally so as a guide to design in this case When applied, its apex will in like manner be at the upper level, or at $H + d$, and be similarly truncated at the crest

The trapezoid thus formed will have a base width of

$$b = \frac{H + d}{\sqrt{\rho}} \qquad (13)$$

which formula will be found approximately correct, as a general rule, and with a top width of $\frac{d}{\sqrt{\rho}}$ its area will be $\frac{H + 2d}{2\sqrt{\rho}} \times H$

The crest width of $\frac{d}{\sqrt{\rho}}$ will, however, be found in practice much too narrow for actual requirements unless the depth of water above the crest or d is exceptionally great The following rule will be found to provide a suitable crest width for most ordinary cases, viz ,

$$c = \sqrt{H} + \sqrt{d} \qquad . \qquad (14)$$

with $\frac{d}{\sqrt{\rho}}$ as a minimum This will also apply to submerged weirs, for instance, in the Narora Weir (Fig 24), H is 10 feet and d is 8 feet whence $c = \sqrt{10} + \sqrt{8} = 3\,2 + 2\,8 = 6$ feet In the insubmerged La Grange

Weir (Fig 29) $H = 120$ and $d = 16$, the crest width would be $11 + 4 = 15$ feet

In many cases, however, the necessity of providing space for falling shutters or for cross traffic during times when the weir is not acting renders obligatory the provision of an even wider crest Damage due to shock from floating logs has also to be guarded against With a moderate width, as obtained by formula (14), a trapezoidal outline has to be adopted in order to give the requisite stability to the section This is formed by joining the edge of the crest to the toe of the base by a straight line, the base width of $H + d - \sqrt{\rho}$ being retained

This is done in Figs 24 and 29 (*post*) When the crest width exceeds the dimensions of formula (14) the face of the crest should drop down vertically till it meets the hypothenuse of the elementary profile, as is the case with the pentagonal profile of dams An example of this is given in Fig 28 of the Dhukwa Weir The tentative section thus outlined should be tested by graphical process, and if necessary the base width altered to conform with the theory of the middle third

(43) In American practice with trapezoidal weir profiles, the width of the weir crest is made some fixed proportion of that of the base To be universally applicable, however, the value of c should be also some function of d, and it is possible that a formula might be devised, which should include all three influencing factors, viz, H, ρ, and d

With values of d up to 10 feet the following formula for base width for a trapezoidal profile will be found closely approximate to requirements, $\frac{c}{b}$ or r being taken as 25

$$b = \frac{H + d}{\sqrt{\rho}} \times \frac{1}{\sqrt{r + 1 - r^2}}, \tag{15}$$

$$\text{or } \rho = \frac{2}{3} \text{ and } r = 25 \quad b = 6\,(H + d) \tag{15a}$$

Formula (15) is strictly applicable to dams of trapezoidal profile, H' being substituted for $(H + d)$ When applied to weirs, the value of c in the ratio $\frac{c}{b}$ is the crest width on the supposition that the weir is a dam whose height $H' = H + d$ The actual crest width at the lower level will naturally be wider than the assumed value of c at the higher level

When a reservoir wall has its crest lowered for a portion of its length it acts both as a dam and a weir Its outline is then usually trapezoidal, when formula (15) for base width will apply equally to both portions The value of b will, however, be barely sufficient for the weir section were it not influenced favourably by the reverse pressure of the tail water

(44) A weir is subject to the effect of the reverse pressure of the tail water down stream, from which a dam is usually exempt

This reverse pressure modifies the position of the centre of pressure on

the base in a manner usually but not invariably favourable to stability As the moments of the horizontal pressure of water on either side of the weir wall vary very nearly with the cubes of their height it is evident that a comparatively low depth of tail water will have but small influence and may well be neglected This is accentuated by the slope given to the face when a vertical back is adopted by which the normal water pressure is given a downward inclination that reduces its capacity for good The small result thus produced is illustrated in the force diagram Fig 23a where the reverse is shown combined with the back pressure

(45) Calculations of the depths of water passing over the weir or rather the height of reservoir level above the weir crest designated by d and of

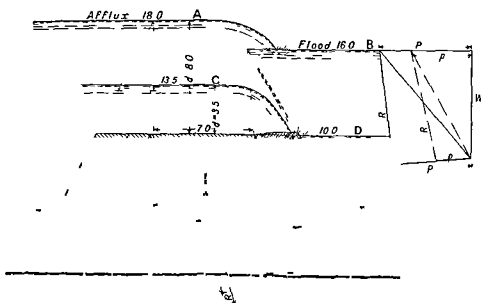


FIG 24 — Narora Weir Wall

its reciprocal depth D in the tail channel are often necessary for the purpose of ascertaining what height of water level up stream or value of d will produce the greatest effect on the weir wall In low submerged or drowned weirs the highest flood level has often the least effect as at that time the difference of levels above and below the weir are reduced to a minimum This is graphically shown in Fig 24 which represents a section of the Narora dwarf weir wall to which reference will subsequently be again made in Chap VI In this profile two resultant pressures R and R^1 are shown in which R^1 due to the much lower water level of the two states under comparison falls nearer the toe of the base

(46) The rise of the river water produces with regard to the stress induced on the weir three principal situations or states which are enumerated below

(1) When the *head water* is at weir crest level, then, except in cases where a water cushion exists, natural or artificial, the tail channel is empty, and the conditions are those of a dam

(2) When the *tail water* is at weir crest level, in this case the reciprocal depth of the head water above crest is found by calculation

(3) At *highest flood level*, the difference between the head and tail waters is then at a minimum (2) Applies only to submerged weirs

In a submerged weir the greatest stress is produced during state (2)

(47) The horizontal water moments on either side of a wall are related to each other in proportion to the cubes of their respective depths In cases where the dividing wall is overtopped and one or both of the water pressure triangles are truncated, the moment of the resulting trapezoid of pressure will be the product of formulas (11) and (12) and will be

$$M = \frac{H^2}{6}(H + 3d) \qquad . \quad . \qquad (16)$$

The moment of the opposing tail water will be, if a triangle, $\frac{D^3}{6}$, or if a trapezoid formula 16 will again apply, d^1 or $(D - H)$ taking the place of d in that expression The difference of the two moments of water pressure up and down stream will then be the balance moment acting on the wall

(48) In order to obtain the proper value of d, during the second " state," it will be necessary to make a calculation of the discharge per foot run of the channel below the weir at the level of its crest

(49) In the case of submerged weirs over rivers with sandy beds, the base of the weir and its floor often coincide with the averaged bed level of the river The river also is of considerable width in proportion to its depth; consequently the formula 100 $Ac\sqrt{RS}$ can be simplified without sensible error by substituting D for R *

As the discharge per unit length, or foot run only, is required, the area A will equal $D \times 1 = D$, and, further, if the friction at the sides of the river be neglected WP, or the wetted perimeter, will equal the base of the water section, *i e*, unity, whence $R = \frac{A}{WP} = \frac{D}{1} = D$ The formula then simplified becomes

$$Q = 100\, cD^{\frac{3}{2}} \sqrt{S} \qquad (17)$$

The coefficient c can be obtained from Jackson's " Hydraulic Manual," Table XII, Part 4, D being taken as R, or from Higham's Tables

(55) Figs 25 and 26 are examples of two low weir walls under similar conditions of water pressure Fig 25 is a vertically backed wall Fig 26 is of equiangular section The bed slope of the river, or S, is assumed at

* A doubtful proposition in practice since the average bed level is liable to vary at different seasons of the year *i e* at different stages of the river s flow, especially when the river is flowing in more than one channel with sand banks between them —ED

1 in 10 000 and when $D = 15$ feet which is the value given to H in the figure the reciprocal depth over crest namely d will be 6 5 (with $n = \cdot 0250$) The resultants are worked out on the supposition that the weir wall is on a

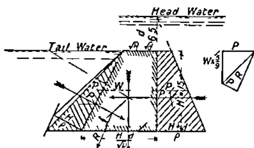


FIG 25

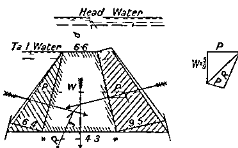


FIG 26

porous sand foundation and is thus subject to loss of weight by flotation This is the almost invariable status of dwarf diversion weirs The incidence of the resultants on their bases go to show that the canting forward of the profile makes no practical difference in the stability of the wall but the

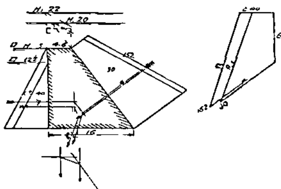


FIG 27

reversal does as shown in Fig 27 which however is under somewhat different conditions

Some examples of high weirs will now be given

(56) The profile of the Dhukwa Weir (Fig 28) is that of a work thrown across the Betwa River in Bundelkund Upper India with the object of forming a second reservoir to supplement the existing one at Parichha the head of the Betwa Canal It was completed in the year 1910 and will be further described with reference to the project and to the weir shutters in Chap VI

In this place it will be sufficient to examine the section only The tail water at full flood does not rise above half the height of the weir consequently as noted in par (46) state (3) of maximum flood will give the correct value of d to be used in the formula (13) $b = \frac{H + d}{\sqrt{\rho}}$ Here $H = 50$ feet and $H + d = 63$ feet The base width should then be $\frac{2}{3} \times 63$ or 42 feet which

Fig 28 —Dhukwa Weir

it scales almost exactly The crest width is necessarily very wide in order to accommodate the high collapsible weir shutters and their gear

In a case like this the flood velocity of approach which must be considerable should be ascertained and the height sufficient to produce this velocity, added to $H + a$, making the formula for base width $\frac{H + d + h}{\sqrt{\rho}}$, and for crest width $\sqrt{H} + \sqrt{d + h}$

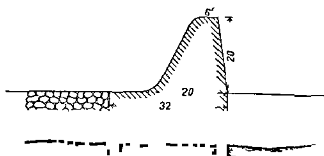
If the velocity of approach were 10 feet a second a not unlikely value in such a river h would equal 1 5 feet This would however only slightly affect the values of b and of c

* (57) The La Grange Weir (Fig 29) is an example of a very high overfall weir situated in California The velocity of the 16 feet deep overfall is esti-

Fig 29 —La Grange Weir

mated to be 13 feet per second Assuming that in the reciprocal channel below to be 6 feet per second the ratio will be 2 1 1 therefore with $d = 16$ feet D will $= 16 + 2 1 = 34$ feet which if measured above the given low water level will hardly reach half way up the weir face Consequently high flood state No 3 of par (46) will apply

(58) The Granite Reef Weir (Fig 31) is another example of an American weir It is founded partly on rock and partly on boulders and sand the section on the latter porous substratum is given in Chap VI as belonging to that class of river weir The superstructure above the floor level is the



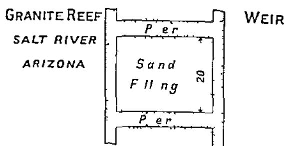


Fig 31

same throughout but the foundations on rock are remarkable as being founded not on the rock itself but on a super imposed cushion of sand Reinforced concrete piers spaced 20 feet apart were built on the bedrock to a certain height so as to clear all inequalities these were connected by thin reinforced concrete side walls the series of boxes formed were then filled up level with sand and the dam built thereon This work was only completed in 1908 and as Schuyler s Reservoirs from which valuable work this account is taken supplies no dimensions the portion of the profile below the floor is conjectural Sand in a confined space is incompressible *and there is no reason why it should not be used in like situations*

The section of the weir is exceptionally strong its base being quite equal to its height

It is an unfortunate fact that in most reference books containing record plans of irrigation works vital statistics such as flood levels discharges

and other matters, are often wanting A work admirable in this respect is the "Madras Irrigation Manual" The work of analysing designs is, therefore, often like making bricks without straw, as not infrequently the most important statistics are entirely unrecorded The too persistent inroad of the photograph is also apt to vitiate the value of reference works The photograph, useful as it may be in presenting an idea of the appearance of a work to the lay mind, can never take the place of a properly dimensioned drawing with the professional student, and the tendency is not to substitute one for the other

(59) Another typical high American gravity weir is illustrated in Fig 32 of the Mariquina Dam, situated in the Philippine Islands The height is

Fig 32 —Mariquina Weir

60 feet, foundation, rock Such a work could very well be erected with perfect safety on a boulder and gravel formation if a floor of suitable dimensions were provided, but the arch buttress type illustrated in the next chapter will be about 40 per cent cheaper

(60) The Castlewood Weir (Fig 33) is a high weir of peculiar construction being formed of dry rubble stonework enclosed in an outer casing of cement masonry There appears to be no reason why a construction of this kind should not answer, but the fact remains that it did not The structure developed serious leaks, probably owing to bad connections with the flanks of the canyon, and had to be reconstructed This was effected by adding an earthen bank with a 3 to 1 slope on the rear of the work, and lengthening the outlet pipe accordingly

(61) An example of a weir with a very high flood passing over is the Folsom Weir on the American River (Figs 34 and 34a) The flood rises to

R L 225 i e, 32 feet above the [illegible] Engineering (Wilson)

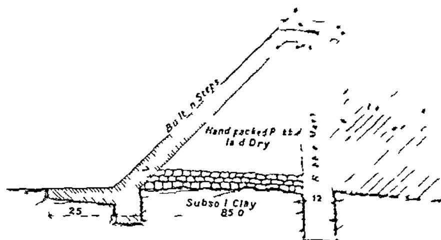


FIG 33—Castlewood Weir

The base width at sill of sluice will be by formula (1 $\frac{2}{3} \times 81 = 54$ feet which is just about what it scales The [illegible] formula (14) should be $\sqrt{70} \times \sqrt{24} = 13\frac{1}{4}$ feet but the minim[illegible]

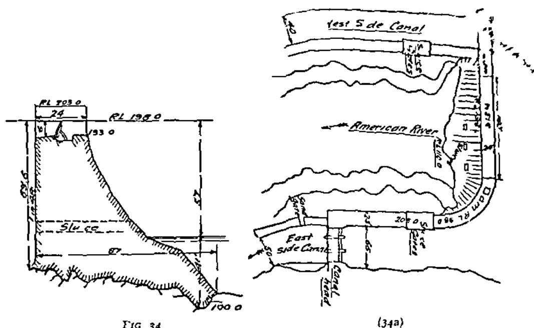


FIG 34

(34a)

FIGS 34 and 34a—Folsom Weir

will be $\frac{2}{3} \times 30 = 20$ feet (par 42) Its exceptional width of 24 feet is due to the hydraulic collapsible shutters which form so excellent a feature in the design

(62) The highest weir in the world is that belonging to the Coolgardie water supply reservoir, near Fremantle, in Western Australia The profile is given in Fig 35 On account of its immense height, this structure is to all intents and purposes a dam The tail water will have very little influence, except detrimentally as regards load on the foundations

The limiting stress λ is 8 tons The base of low weir and the elementary triangular profile have been dotted on the section It is considered that the rear projection of the heel is excessive, a more nearly vertical back would be statically better If the crest were thrown forward to reduce the ore

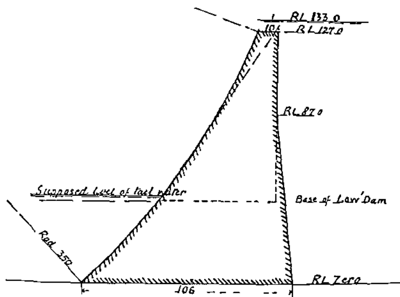


FIG 35 —Coolgardie Reservoir Weir

slope and allow the water to fall clear of the face, the sectional area would have to be somewhat increased

(63) Canal falls and tank escape weirs and works of this description subjected to a fixed and moderate depth of water, if built in solid clay cutting well backed with puddle, are not considered as subject to full hydrostatic pressure as would be a wall without such solid backing, consequently the base width can be reduced to a mean between that of a retaining wall for earth and one for water This is effected by making the base width not $\frac{H+d}{\sqrt{\rho}}$, as in formula (13), but

$$b = \frac{H+d}{\rho}; \quad \ldots \quad \ldots \quad (18)$$

with $\rho = 2\frac{1}{4}$ this is equivalent to $b = \frac{4}{9}(H+d)$, and with $\rho = 2$ this is equivalent to $b = \frac{H+d}{2}$

This is termed the "Hybrid" section, and is further noticed in Chap IX

Editor's Notes

(D) *Foundation Pressures*, pars (7) *et seq*

It is a fundamental principle of the design of high dams of masonry or concrete that they must be founded on hard, unyielding rock, relatively impermeable by water and proof against disintegration by percolation. Another fundamental principle is, as the author has pointed in par (4), that the profile of the dam must be such as will contain the lines of resultant pressure within the " middle-third " of its thickness In the circumstances, the question of the effect on the foundation soil of resultant pressures falling outside the middle-third, cannot arise It is different in the case of designs for retaining walls, which, as a rule, have to be built on ordinary soil—often on very soft, yielding soil—and in which, according to current conventions, it is correct to contemplate the resultant pressures falling far outside the middle-third Our author has, however, in pars (7) *et seq*, and notably in pars (23), (24), (25), and (26), entered into a discussion of the theory regulating such eccentric pressures, in Chap II, instead of in Chap I The formula used by him in par (7), viz

$$\text{maximum stress} = \text{mean stress}\left(1 \pm \frac{6c}{b}\right)$$

holds good only so long as the whole stress is positive, but when part of the stress is negative implying tension, instead of compression, the factor b is reduced to something less than the full base-thickness, and mean and maximum pressure intensities are increased Thus, the formula holds good in the cases depicted in Figs 5 and 7, Chap II, and in stress diagrams R and R^2 of Fig 6, but not in stress diagram R' of Fig 6

In this last-named case the author infers that two-thirds of the base will be in compression when the resultant thrust impinges on the extreme toe of the base and when, therefore, presumably, the wall is just on the point of balance to overturn That seems to be obviously an erroneous idea

He is considering the conditions of a dam exerting a total vertical pressure of 300 tons on a base 66 7 feet thick When this pressure is distributed evenly over that base its intensity is 4 5 tons per square foot But in Fig 6, stress-diagram R', only two-thirds of the base, or 44 4 feet, is supposed to be in compression, which implies that the mean intensity is 6 75 tons per square foot, varying from zero at the $\frac{2}{3}$ point, up to 13 5 tons per square foot at the toe For some unexplained reason the author, in par (26), assumes the mean pressure intensity in Fig 6 to be 4 tons per square foot instead of 4·5, and, even so, his diagram R, shows a compression area of $\frac{16 \times 44\ 4}{2} = 355$ tons, although the weight of the dam and its load is only 300 tons For the reason given by the author in par (17), if, in case R' of Fig 6, only two-thirds, or 44 4 feet of the base were in compression, the centre of pressure would be two-ninths, or 14 8 feet distant from toe of base, and not at the extreme toe-edge, as shown in Fig 6

When the centre of pressure falls outside the middle-third of the base so that a portion of the base is under tension, the portion under compression is reduced from b to b', the mean intensity of pressure is increased to $\frac{W}{b'}$,

and the deviation of the centre of pressure c' from the centre of base is measured from the centre of b' instead of from that of b. The formula for maximum stress becomes changed then from $\frac{W}{b}\left(1 \pm \frac{6c}{b}\right)$ to $\frac{W}{b'}\left(1 \pm \frac{6c'}{b'}\right)$.

In the case depicted in Chap. I, Fig. 6, the total vertical pressure is 17.5 tons which distributed over 12 feet of base gives a mean of 1.46 tons per square foot. According to our author's reckoning the centre of pressure falls 1.25 feet inside the toe of the base. If that were true it would mean that only $1.25 \times 3 = 3.75$ feet length of the base would be in compression, the mean pressure on which would be $\frac{17.5}{3.75} = 4.67$, and the maximum at toe would be 9.34 tons per square foot, an intensity which no soil except hard rock could be relied on to support.

CHAPTER III

SECTION I —ARCHED DAMS (TYPE B)

(1) In this type, designated as B, the whole dam, being arched on plan, is supposed to be in the statical condition of an arch under pressure. As, however, the base is immovably fixed to the foundations by the frictional resistance due to the weight of the structure, the lowest portion of the dam cannot possess full freedom of motion or elasticity, and consequently must act more or less as a gravity dam subject to oblique pressure.

However this may be, experience has conclusively proved that if the profile be designed on the supposition that the whole is an elastic arch, this conflict of stresses near the base can be neglected by the practical man. The probability is that both actions take place, the true arch action, at the crest, gradually merging into transverse stress near the base, the general result being that the safety of the dam is enhanced by this combination of tangential and vertical stresses on two planes.

(2) In this type of structure, the weight of the arch itself is conveyed to the base producing stress on a horizontal plane, while the water pressure normal to the extrados, radial in direction, is transmitted through the arch rings to the abutments. The pressure is therefore distributed along the whole line of contact of the dam with the sides as well as the ground. In a gravity dam on the other hand the whole pressure is concentrated on the horizontal base.

(3) The average unit stress developed by the water pressure is expressed by the formula

(Short Formula) . . $s_1 = \frac{RHw}{b} \qquad b = \frac{RHw}{s_1}$. (1)

in which R is the radius of the extrados sometimes measured to the centre of the crest, H the depth of the lamina, b its width and w the unit weight of water. In this formula, ρ, the specific gravity of the material in the arch, forms no factor. This simple formula answers well for all arched dams of moderate base width. When, however the base width is considerable as, say, in the case of the Pathfinder Dam the use of a longer formula giving the maximum stress (s) is to be preferred. This is derived from the same principle affecting the relations of s and s_1 or the maximum and average stresses, already referred to in Chap. II on Gravity Dams pars (7) and (17). The expression is in terms of R and b

(Long Formula) . . . λ or $s = \frac{2\,Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)}$ (3)

also $b = R\left(1 - \sqrt{1 - \frac{2Hw}{\lambda}}\right)$ (4)

(4) In a manner similar to type A, the theoretical profile suitable for an arched dam is a triangle having its apex at the extreme water level, its base width being dependent on the prescribed limiting pressure. Successful examples have proved that a very high value for s (or λ), the maximum stress, can be adopted with safety. If it were not for this the profitable use of arched dams would be restricted within the narrow limits of a short admissible radius, as with a low limit pressure the section would equal that of a gravity dam.

(5) The water pressure on an arch acts normally to the surface of its back and is radial in direction; consequently the true line of pressure is the arch ring corresponds with the curvature of the arch and has no tendency to depart from this condition. There is therefore no point of rupture as in the case in a horizontal circular arch subjected to vertical, not radial pressure. This property conduces largely to the stability of an arch under liquid pressure. *This condition is not strictly applicable in its entirety to the case* of a segment of a circle held rigidly between abutments, as the arch is then partly in the position of a beam. The complication of stress involved is, however, too abstruse for practical consideration.

(6) As we have already seen, the correct profile of the arched dam is a triangle modified into a trapezoid with a narrow crest, and with regard to arch stresses the most favourable outline is that with the back or extrados vertical. The reason for this is that the vertical stress due to the weight of the arch, although it acts on a different plane to the tangential stresses in the arch ring, still has a definable influence on the maximum induced stress in the arch ring. This vertical pressure produces a transverse expansion which may be expressed as $W \times E \times m$, in which E is the coefficient of elasticity of the material and m that of transverse dilatation. This tends, when the extrados is vertical, to diminish the maximum unit stress in the section, whereas when the intrados is vertical and the back inclined, the modification of the distribution of pressure is unfavourable, the maximum stress being augmented in similar degree. When the triangular profile is equiangular an intermediate or neutral condition exists. A profile with vertical extrados should, therefore, be adopted whenever practicable.

(7) In very high dams, however, the pressure on the horizontal plane of the base, due to the weight of the structure, becomes so great as to even exceed that in the arch ring; consequently it is necessary to adopt an equiangular profile in order to bring the centre of pressure at or near to the centre of the base, so as to reduce the ratio of maximum pressure, and average pressure, to a minimum. As stated in the previous chapter, par. (7), when a vertical through the CG of the profile passes through the centre of the base, the maximum pressure equals the average or $= \frac{W}{b}$.

(8) When the back of an arched dam is inclined, the weight of the water superimposed is carried by the arch, and does not reach the base, but the

abutments The unit pressure induced in the arch ring is clearly the same whatever be its inclination to the vertical, for this reason, that the liquid pressure never exceeds Hw (H being any vertical depth), and is radial and normal to the back in direction The area of water pressure acting on an inclined, is naturally greater than that acting on a vertical surface, but the length of the surface in question is greater in similar degree, so that the unit stress in the arch ring remains unaltered, the total stress conveyed to the buttresses or abutments, is, however, naturally greater with an inclined than with a vertically backed profile This does not affect the problem we are now considering but when the dam consists of a series of vertical arches with intervening buttresses as in type C this question is one that demands attention Some examples of arched dams will now be exhibited

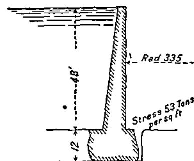


Fig 1 —Bear Valley Dam

* *Bear Valley Dam* (Fig 1)

(9) This small work is the most remarkable example of arched dam in existence, and as a precedent is most valuable, the pressure induced in the arch ring just above the widening works out as 53 tons per square foot The section would be better reversed This proves the capability of this type of dam to stand a pressure previously deemed impossible

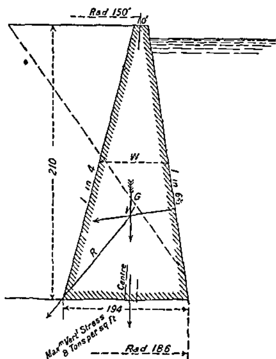


Fig 2 —Pathfinder Dam

Pathfinder Dam (Fig 2)

(10) This immense work has a radius of 150 feet measured to the centre of the crest, that, however, of the extrados at the base of the profile is 186 feet, and this quantity has to be used for the value of R in the long formula No 3 The unit arch stress then works out to 11 8 tons The average vertical stress on the base is 7 tons and the maximum about 8 If the profiles were altered to one with a vertical back, the arch stress would be less, as the radius would be reduced to 155 feet, but that on

* See Editor s Note at end of this chapter —Ed.

the base would be nearly double the average, viz , 14 tons It is deemed that the existing outline is the better of the two

(11) The Shoshone Dam (Fig 3) is designed on identical lines to the last example It has the distinction of being the highest dam in the world

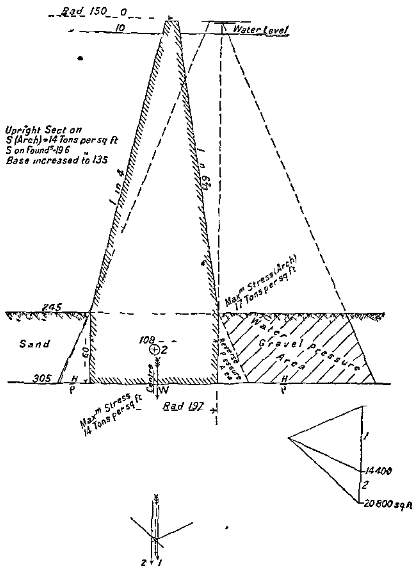


FIG 3 —Shoshone Dam

The value of s (vertical stress) is 14, and that in the arch 17 tons respectively, at the 245 foot depth If the profile were altered to a vertically backed trapezoid, as shown dotted on the diagram, and the base extended at the toe, the superficial area would be increased from 21,000 to 22,000 square feet The average vertical pressure would then be 10 tons and the maximum 19 6 tons, while the arch pressure will be 14 tons, the reduction in arch stress below the previous being due to the shortening of the radius at the base

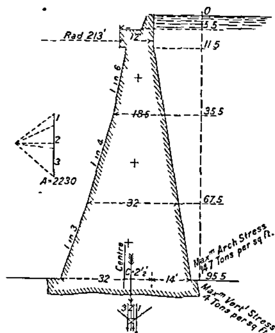


FIG. 4—Sweetwater Dam

This proves the soundness of the inclined backed profile which has been adopted

In both these dams a crest width of 10 feet has been adopted

(12) The profile of the Sweetwater Dam in California is given in Fig 4. This work, during a great flood was overtopped by 5 feet of water and

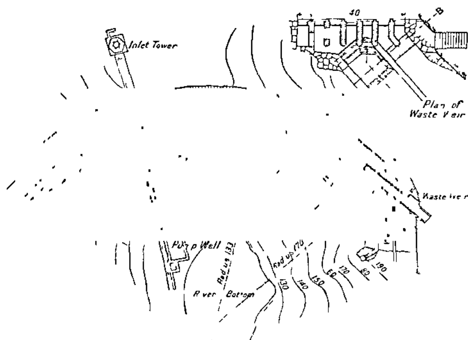


FIG 4a—Plan of Sweetwater Dam

subsequently the crest was raised by a thin wall up stream to prevent its recurrence The maximum unit stress developed works out by the long



Fig 4b —Photograph of Sweetwater Dam.

formula 3, par. (3), as follows :—The radius of the extrados at the base being 232 feet, $\frac{b}{R} = \frac{46}{232} = \cdot 2$. The expression then becomes by the "Long"

formula

$$s = \frac{2 \times 95 \times \frac{1}{16}}{\cdot 2\,(2 - 2)} = 14\ 7 \text{ tons}$$

The back is canted forward with the object of equalising the stress on the base With a comparatively low height, as in this case, a slighter rear batter or a perfectly vertical back would answer better—for, as we have seen, the inclined back is not of advantage to the arch, throwing as it does a weight of water on the dam which otherwise would be avoided The Sweetwater is a comparatively old work The maximum unit pressure due to the vertical weight is obtained by using formula (2) par (7) Chap II W being $= A \times \omega\rho$, or, in figures, $2\ 240 \times \frac{1}{16} = 140$ tons And c measuring 2 5 feet, the stress works out as below —

$$\frac{W}{b}\left(1 + \frac{6c}{b}\right) = \frac{140}{46}\left(1 + \frac{15}{46}\right) = 4 \text{ tons nearly}$$

This is low unit stress, while that in the arch is high With a vertical back the former would be increased, which it can well afford, and the latter be

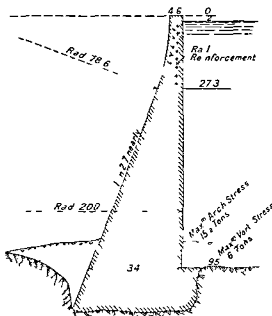


FIG 5 —Barossa Dam

reduced to 14 tons owing to the shortening of the radius of the extrados—besides being otherwise in a better statical position (*vide* par (6)) For the plan photographic view of this dam given as Fig 4b we are indebted to Schuyler s Reservoirs '

(**13**) The Barossa Dam (Fig 5) is an Australian work

The back is vertical and the fore batter is almost 1 in 2 7 The outline is not trapezoidal but pentagonal viz , a square crest imposed on a triangle—

the face joined with the hypothenuse of the latter by a curve The crest is slender, being only $4\frac{1}{2}$ feet wide but is strengthened by rows of 40 lbs iron

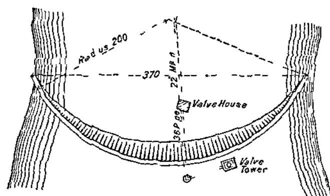


Fig 5a —Site Plan

rails, fished together built into the concrete The maximum arch stress works out to $15\frac{1}{2}$ tons, the corresponding vertical stress on base to 6 tons

Fig 5a is a site plan of the work, in which the wavy lines on the flanks represent earthen slopes not water.

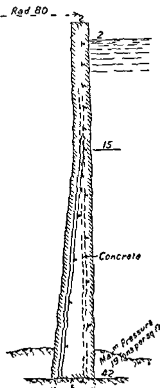


Fig 7 —Barren Jack Dam

(14) Arched dams abut either on the solid rock banks of a canyon or else on the end of a gravity dam In cases where a narrow deep central channel occurs in a river, this portion can advantageously be closed by an arched dam, while the flanks on which the arch abuts can be gravity dams aligned tangential to the arch at each end The dam will thus consist of a central arch with two inclined straight continuations The plan of the Roosevelt Dam (par (38), Chap II) will give a good idea of this class of work

(15) Fig 7 is the profile of a temporary reinforced arched dam for domestic water supply at Barren Jack, Australia The reinforcement consists of iron rails The arch pressure at the base works out to 19 tons nearly

Section II —Arch Buttress Dams (Type C)

(16) In this type, the dam is composed of a series of vertical or inclined arches divided into spans by buttress piers The arrangement is in fact identical with that of a masonry arched bridge, if the latter could be considered turned over on one side

The following estimate has been made of the economy in cost of this

type in comparison with the gravity type of dam, under the same condition of limiting stress —

At	30 feet in height, saving is	31	per cent
,,	50 ,, ,,	28	,,
,,	100 ,, ,,	23	,,
,,	150 ,, ,,	12	,,
,,	200 ,, ,,	9	,,

In a design at the end of this chapter for a Dam of type C, 64 feet in height the saving in material works out to 50 per cent, so that by allowing proper high stresses for arches under liquid pressure and by adopting large spans the actual saving effected will, it is believed considerably exceed that given above

(17) The first example will be that of the Mir Alam Tank dam (Fig 8) This work is situated near Hyderabad, the capital city of the Nizam's dominions in the Deccan South India

This remarkable structure is quite unique of its kind and forms a most instructive example of the principles governing the design of this type of dam It was built over a century ago by some engineer of "John Company," *whose name has not been handed down by fame to posterity* The dam, which is aligned on a wide curve consists of a series of vertical semi-circular arches of various spans These arches abut on buttress piers The spans vary in width from 70 to 147 feet one of which latter is shown in Fig 8 The height is 40 feet Water has been known to overtop the crest

On account of the inequality of the spans the adoption of the semi-circular form of arch is evidently a most judicious measure, for this reason, that an arch of this form under liquid pressure exerts no lateral thrust at the springing The water pressure, as already noted in the previous section, is radial in direction consequently the half arch is balanced and in equilibrium Whatever thrust is exerted is not in the direction of the axis of the arch, but on that of the buttress piers, *i e*, at right angles to the springing line On the other hand, if the arch were segmental in outline, as is shown dotted on Fig 8 the tangential thrust is intermediate between the two axes, and when resolved in both directions one component of the thrust acts along the axis of the dam This has to be met either by the abutment, if it is the end span, or else by the corresponding thrust of the adjoining half arch The other component is carried by the buttress, therefore, if segmental arches are used in order to avoid inequality of thrust the spans must be equal

The whole work is built of brickwork, but the unit stress in the arch ring at the base, using the short formula $s_1 = \frac{RHw}{b}$ par (3), works out to over $10\frac{1}{2}$ tons This dam therefore forms a useful precedent as proving how high a stress can be borne with safety by an arch of such decidedly inferior material as brickwork in ordinary lime mortar The arch is vertical and $8\frac{1}{2}$ feet thick throughout, and 40 feet high

The buttress piers in this work are very short projecting only 25 feet beyond the spring line of the arches and being altogether only 42 feet long This length and the corresponding height would clearly be inadequate to

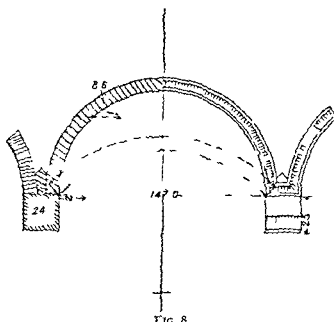
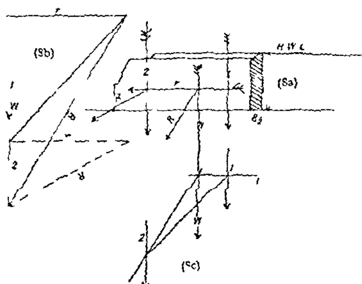


Fig 8



Figs 8 8a 8b 8c—Mir Alam Arched Panel Dam

withstand the immense horizontal thrust which is equivalent to $\frac{H^2}{2} \times w \times l$ $= \frac{1600 \times 173}{2 \times 36} = 3844$ tons This is proved in Fig 8a where the resultant stress line on this supposition falls just outside the toe of the base and has a dangerously low inclination to the horizontal It is evident that if the

buttress pier slides, or overturns, the arches behind it must follow suit, for which reason the two half arches and the buttress pier cannot be considered as separate entities, but as actually forming one whole, and consequently it follows that the effective length of the base must extend from the toe of the buttress right back to the extrados of the two adjoining arches At, or a little in rear of the spring line, the base is split up into two forked curved continuations The weight of these arms, belonging to the adjoining arches, has consequently to be included with that of the buttress proper when the stability of the structure against overturning or sliding is estimated

(18) In the transverse section (Fig 8a), the graphical calculations establish the fact that the resultant line intersects the base thus lengthened almost exactly at its centre, the direction of the resultant R is also satisfactory as regards the angle of frictional resistance The maximum unit pressure in the masonry of the pier is $\frac{R \sec \theta \times w\rho}{\text{Area of base}}$ (par (8), Chap II), which works out to 3·4 tons only. In estimating A, or the area of the base, the length scales 107 feet, but the width of the buttress portion is 25 feet, while that of the two branches combined is 17 feet The greater curved length of the latter, however, if allowed for, would enlarge the area by about $\frac{1}{4}$, which would be equivalent to increasing the width to 21 feet Therefore 22 feet has been adopted as an average representative width for the whole base The actual figures are $\frac{14{,}300 \times 2}{36 \times 107 \times 22} = 3{\cdot}4$ tons The specific gravity is taken as 2 The quantity 14,300 is the value of N_1, or of $R \sec \theta$, not shown in Fig 8b The incidence of the resultant actually falls within the two arches, not in the buttress proper

(19) The design could be much improved by altering the distribution of the material of the section of the arch, by making it trapezoidal instead of rectangular in profile This is effected by increasing the base and decreasing the crest width Thus, for the same sectional area, a much stronger profile could be produced subject also to nearly half the unit stress to which it is liable in its present form The base could be widened from $8\frac{1}{2}$ to 14 feet, and the crest reduced to 3 feet or $\frac{\sqrt{H}}{2}$ par (11) The maximum unit stress would then be lowered from 10 6 to under 6·5 tons.

(20) If the arch were altered on plan from a semicircle to a segment of a circle, the radius would of necessity be increased, and the stress with it, a thicker arch would therefore be required This would not quite compensate for the reduced length of arc, but on the other hand, owing to the crown being depressed, the effective base width is reduced and will have to be made good by lengthening the buttress piers What particular disposition of arch and buttress would be the most economical is a matter which can only be worked out by means of a number of trial designs

(21) There are not many modern examples of arch buttress dams The Mir Alam Dam has remained resting on its laurels without a rival for over 100 years Fig 9 is an early example of a segmental panel arch dam It is of the Belubula Dam in New South Wales The arch crest is 37 feet above base very nearly the same as in the last example The arches which are inclined 60° to the horizon are built on a high solid platform which obliterates inequalities in the rock foundation This platform is 16 to 23 feet high so that the total height of the dam is over 50 feet The spans are 16 feet with buttresses 12 feet wide at the spring line tapering to a thickness of 5 feet at the toe they are 40 feet long The buttress piers which form quadrants of circles in elevation diminish in thickness by steps from the base up these insets corresponding with similar ones in the arch itself These steps are not shown in the drawing the arch also is drawn as if in one straight batter The arches are elliptical in form and the spandrels are filled up flush with the crown presenting a flat surface towards the water

(22) Some of the features of this design are open to objection Firstly the filling in of the arch spandrels entirely abrogates the advantage accruing to arches under liquid pressure The direction of the water pressure in this case is not radial but normal to the rear slope thus exactly reproducing the statical condition of a horizontal arch The pressure therefore increases from the crown to the haunches and is parabolic not circular in curvature The arches should have been circular not elliptical with the spandrels left empty to allow of the radial pressure which is so beneficial Secondly the stepping in of the intrados of the arch complicates the construction A plain batter would be easier to build particularly in concrete Thirdly the tapering of the buttress pier towards the toe is quite indefensible the stress does not decrease but increases towards the toe being a maximum at the edge of the section The whole structure is liable to overturning moment which moment is resisted by its base so that any reduction of area at the toe of the buttress accentuates the unit stress at the most critical point

(23) The inclination of the axis of the arch to the vertical is generally a desirable in fact a necessary feature wherever segmental arched panels are used the weight of water carried is of value in depressing the final resultant line to a suitable angle having regard to frictional resistance to shearing stress As noted in Section I the weight of the water overlying the arch does not increase the unit stress in the arch ring Consequently any inclination of axis can be adopted without in any way increasing the unit stress due to the water pressure This is due to the radial and normal direction of the pressure which causes it to be all taken up by arch stress When however the arch is inclined a certain proportion of the weight of each lamina in which it may be considered as divided is conveyed by arch action to the abutments In the diagram (Fig 9a) the vertical load line W represents the weight of one unit or one cubic foot of the arch ring which is equal to wp This force is resolved in two directions one p parallel to the axis of the arch and the other n normal to the former The force $n = W \sin \theta$ θ being the

inclination of the arch axis to the vertical, $p = W \cos\ \theta$. The unit stress developed by the radial force n is similar to that produced by the water

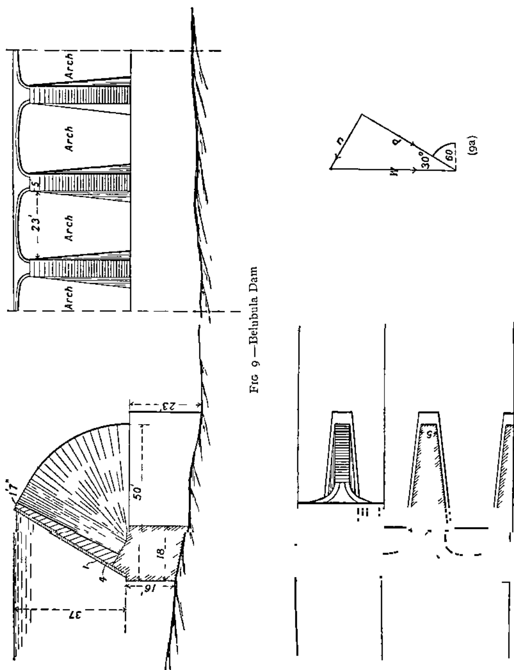


FIG 9—Belubula Dam

pressure which is also radial in direction and is R_1n, but R_1 the radius in this case, is the mean radius, the pressure being internal, not external The unit stress s will then be

$$s = R_1 n p \sin\ \theta \qquad (6)$$

When θ is 30 degrees, $\sin\ \theta = \frac{1}{2}$, when 45 degrees $= \sin\ \theta = 0\ 707$

It will easily be understood that this unit stress due to u does not accumulate, but is the same at the first foot depth of the arch as it is at the bottom, the width of the lamina also does not affect it The component p does, however accumulate, and the expression $u\rho \cos \theta$ should be multiplied by the inclined height h lying above the base under consideration As $h = H \sec \theta$, the unit compressive stress at the base will be $\frac{Hu\rho \times b^1}{b}$ in which b^1 is the mean width of the arch If the arch were a rectangle, not a trapezoid, s would $= Hu\rho$ simply

(24) Fig 10 is a design for a segmental arch panel dam, or, rather, weir The height of the crest is 64 feet above base, with 5 feet of water passing over, the apex of the triangle of water pressure will then be 69 feet above the base The inclination given to the axis which is coincident with the spring line and the intrados, is 60° with the horizon

In designing such a work, the following salient points first require consideration —

Firstly Width of span This, it is deemed, should for economical reasons be never less than the height of crest above base In the Mir Alam Dam the span is over three times the depth of water upheld In the present case it will be made the same, that is 64 feet

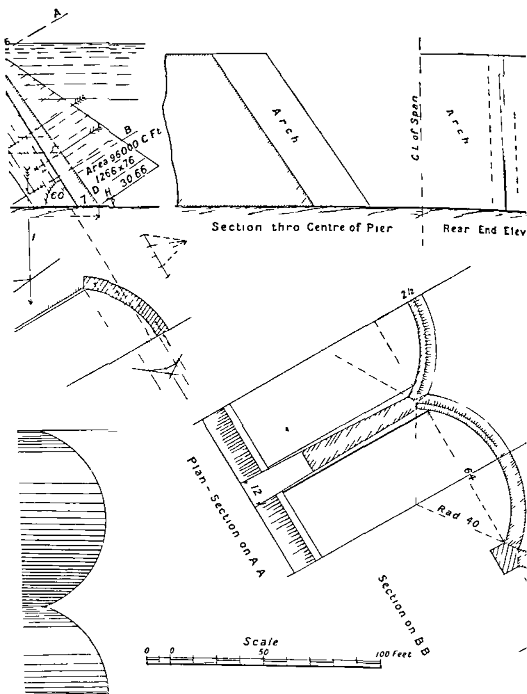
Secondly *Thickness of buttress piers As with bridge piers the width* should be at least sufficient to accommodate the skew backs of the two arches, and a width of 12 feet or about $\frac{3}{4}$ span will effect this

Thirdly Radius and versed sine The radius will be made 40 feet, this allows of a versed sine of $\frac{1}{4}$ span or 16 feet, which is considered a correct proportion to afford a good curvature

Fourthly Thickness of arch This must first be assumed, as its thickness depends on R, the radius of the extrados as well as on the value assigned to λ the limiting pressure This latter will be fixed at $12\frac{1}{2}$ tons a value by no means excessive for arches under liquid pressure With a base width of 7 feet, the radius of the extrados will be 47 feet The base will be considered, not at the extreme depth of 64 feet below crest, but at the point marked D, *where a line normal to the inclined intrados at its base cuts the extrados* of the arch H will therefore be 60 feet The stress due to the water pressure, using the short formula (3) par (3), will be

$$\frac{RHu}{b} = \frac{47 \times 60 \times 1}{7 \times 36} = 11\ 2 \text{ tons}$$

To this must be added that due to the weight of the arch ring from formula (6), par (23) $s = \frac{R_1 u\rho}{2}$ (the angle θ being 30° and $\sin \theta = \frac{1}{2}$), which will be in figures $\frac{43\ 5 \times 1}{2 \times 16} = 1\ 36$ tons, the total stress being a trifle over $12\frac{1}{2}$ tons The 7 feet base width will then be adopted The depth of water producing this pressure is taken as 60 not as 65 feet, which is $H + d$, the *reason being that the reverse pressure due to the tail water, which must at*



Design for Arch Buttress Dam.

least be level with the water cushion bar wall, will reduce the effective depth to 60 feet

(25) The reverse pressure has a great influence in the case of calculations based on the hydrostatic pressure alone, the head being the difference of levels—above and below, whereas, where overturning moment is concerned, the balance moment (*vide* formula (16), par (47), Chap II) is $M = \frac{H^2}{6}(H + 3d) - \frac{h^3}{d}$ or taking the base of both triangles of pressure at the level of D, $M = \frac{3,600}{6}(60 + 15) - \frac{125}{6} = 45,000 - 21 = 44,979$ feet The relation of the reverse to the up stream moment is then 1 2,143, whereas the reverse pressure which affects the thickness of the arch is as 5 65 or 1 13

This question of the great divergence between simple hydrostatic pressure and overturning moment produced by the same head or difference of levels is one often lost sight of Works are stated to be subject to a certain head of water pressure which, unless the depths up stream and down stream are known, can give no correct impression of the effect produced It would do so in the case of a floor on a porous foundation exposed to upward hydrostatic pressure, but not so when the horizontal pressure of water acting against a surface has to be considered

(26) The crest width of the arch according to formula (5), par (11), should be $\frac{\sqrt{H}}{2} = 3\frac{3}{4}$ feet nearly It will be made 3 feet, with a stiffening rib or rim of 2 feet in width It could well be made a fraction of the base width, say $= 3b$, and be reinforced wherever the thickness falls below 2 feet

The length of the pier base is measured from the extrados of the arch, the two half arches forming, as already explained in par (17), a forked continuation of the buttress pier base

The battering of the sides of the pier would clearly be a correct procedure, as the pressure diminishes from the base upwards A combined batter of 1 in 10 is adopted which leaves a crest width of 5 6 feet The length of the pier base, as also its outline were determined by trial graphical process, with the object of manœuvring the centre of pressure as near that of the base as possible, so as to equalise the maximum and the mean unit stress as much as possible This has been effected as shown by the incidence of the final resultant on the elevation of the buttress pier

(27) *Pressure on foundations*

The total imposed weight is measured by N in the force diagram and amounts to 150 000 cubic feet of masonry which at a specific gravity of $2\frac{1}{4}$ is equivalent to $\frac{150,000}{16} = 8,138$ tons The average pressure is this quantity divided by the area of the base, or by $125 \times 12 = 1,490$ super. feet,

the quotient being 5½ tons nearly The maximum pressure will be the same owing to the incidence of R at the centre of the base * It will be noticed that A greatly exceeds B This is due to the added weight of water represented by the inclination given to the force line P, which represents the water pressure

According to the force diagram R^1 scales 174 000 cubic feet or 10 800 tons Dividing this by the area of the base or by 1 490 super feet the quotient 7 2 tons is the average or mean unit stress the maximum being the same The cubic contents per foot run works out to $\frac{64\,000}{76} = 850$ cubic feet nearly, but if the stress be increased to 10 tons in the buttress this will be reduced to 800 cubic feet

The contents of a gravity weir with base width $_3(H + d)$ and top width $\sqrt{H} + \sqrt{d}$ works out at 1 728 cubic feet, the saving in material is therefore over 50 per cent

(28) The Ogden Dam Fig 11, lately erected is a notable example of the arch and buttress type C Its height is 100 feet that is in excess of anything

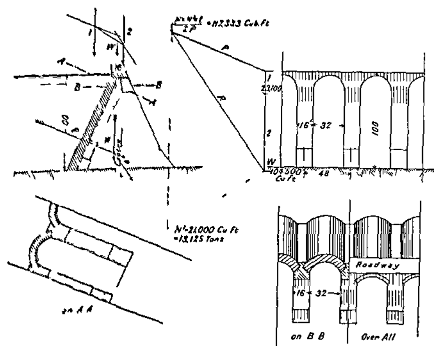


FIG 11 —Ogden Dam Utah

hitherto attempted The inclination of the arches is less than ½ to 1 or about 25° to the vertical The profile of the buttress is equiangular except for a small out throw of the toe On the whole it must be pronounced a good design but could be improved in the following points the arch is

* This stress is an excessive intensity of pressure for ordinary firm earth or for anything weaker than sound rock Rock soil of some hardness would be needed also to withstand disintegration by the water spilling over the crest —ED

unnecessarily thick at the crest and could well be reduced from 6 to 2 feet, thus effecting considerable economy The designers were evidently afraid of the concrete in the arch leaking, and so overlaid the extrados with steel plates The finish off of the crest by another arch forming a roadway is an excellent arrangement, and is well suited for a dam, for a weir, on the other hand, the curved crest is preferable from the increased length of overflow provided The stress diagram shows that the value of N_1 is 13,125 tons The incidence of R on the base is 5 feet from the centre, consequently $S = \frac{N_1}{A}\left(1 + \frac{6c}{b}\right) = \frac{13\,125}{110 \times 16}\left(1 + \frac{3}{11}\right) = 8\,9$ tons (*vide* formula (2), par (7), Chap II)

The pressure on the arch ring at the base by the short formula works out by $\frac{RHw}{t}$ to $\frac{24 \times 100}{8 \times 36} = 8\frac{1}{3}$ tons

The contents of the dam per foot run amounts to $\frac{104{,}500}{48} = 2{,}177$ cubic feet, that of a gravity dam would be about 3 500 cubic feet per foot run, making a percentage in favour of the arched type of nearly 30 Actually the saving amounted to only 12 per cent, this was owing to the steel-covering

Reinforced Concrete Flat Deck Dams and Weirs (Type D)

(29) Of late years an increasing number of dams and weirs of the panel and buttress reinforced concrete type have been constructed of heights, it is believed, up to 100 feet These are all on much the same pattern This consists of a deck sloping at 45°, supported by buttress piers at short intervals These piers are nearly triangular in profile, having their apex at

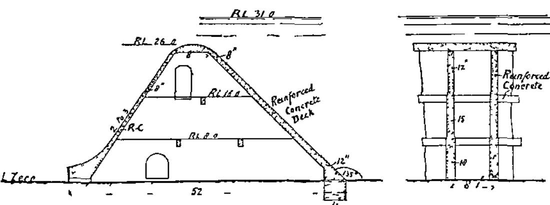


Fig 12 —Schuylerville Weir

a point above crest level On the down stream slope a thin covering is often placed over the ends of these piers, providing a rollway for water if a weir The whole thus forms a hollow box with numerous partition walls The space thus provided is utilised in some cases to accommodate the power plant and to afford cross communication In the former case the partition walls are

FIG 13

(13a)

not contınuous across, but form an arcade, affordıng space for the machınery. The desıgns are admırable, the only poınt remaınıng to be decıded ıs whether

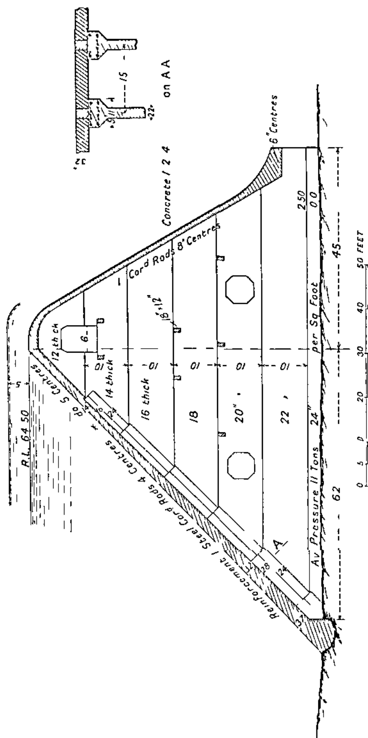


FIG 14 —Ellsworth Weır

they can be ımproved upon ın poınt of cost by substıtutıng arches for the reınforced flat panels and convertıng the profile ınto type C. The use

of a flat deck demands considerable reinforcement, as well as frequent supports

(30) An example of a small work in type D is given in Fig 12 of the Schuylerville Weir, while an alternative design in type C has been prepared in Fig 13 and 13a The arrangement is similar to the last example (Fig 9), except that the piers are hollow No doubt a wider span than 20 feet would give better results

The comparative quantities per foot run are as follows —

Fig 12	Reinforced concrete	215 cubic feet per foot run
Fig 13	Plain concrete	178 ,, ,, ,,

This is considerably in favour of type C

On the other hand, a direct overfall is provided in the last, not a rollway although the direct overfall is considered the better style, barring ice floes *

(31) Another much larger section is given in Fig 14, that of the Ellsworth Weir This work is of the same height as Fig 10, which was designedly made so for the sake of comparison

The comparative quantities per foot run of these two can now be set out

Fig 14 Reinforced concrete, 11,000 cubic feet per 15-foot bay, equivalent to 733 cubic feet per foot run

Fig 10, as by par (27), 800 cubic feet per foot run (with *S* raised to 10 tons)

The advantage in actual quantity of concrete lies with Fig 14, but when the cost of the steel, which amounts to 8½ tons per 15 foot bay together with the extra expense involved in construction and in the quality of the concrete, is taken into consideration the advantage will be with the arched type

EDITOR'S NOTES

(E) Par (9), Fig 1, *Stresses in Arched Dams*

The old Bear Valley Dam, of Redlands (Cal U S A), which is depicted in Fig 1, was built in the year 1884 for the syndicate of a local irrigation enterprise Its total height above foundations is 64 feet, and it was designed with a base-thickness of 20 feet, but (as the story goes) a crisis of financial stringency arose after the dam had been built up 16 feet only, and the engineer was faced with the alternatives of either abandoning the project or of building the dam, from stream-bed level, upwards, to a much slenderer section than he would otherwise have considered advisable Hence the curious appearance of the profile The dam was curved, in plan, to a radius of 335 feet, and its arc was 300 feet long at top It stored 1 742 million cubic feet of water for the irrigation of the lovely and productive orange groves of Redlands The desirability of storing larger water supplies for extensions of these orchards led to a project for a larger reservoir formed by a higher dam The existing dam had shown no sign of weakness, but the engineers were afraid to tempt Providence by building it up higher They

* But the direct overfall will require a rock soil to resist the force of the falling water This will not be so necessary in the Fig 12 type —ED

therefore decided to face the extra cost of building an entirely separate new dam at a site about 200 feet down stream of the old one This new dam was built to the design of John S Eastwood, of San Francisco, in the year 1911 It is a " multiple arch " dam of concrete, somewhat similar in type to those depicted in Figs 9, 10, 11 of this chapter It has 10 arches supported by 11 buttresses, the latter being 32 feet apart, centre to centre The dam is 91·5 feet high above foundations and 350 feet long at crest The buttresses are 18 inches wide at top and battered $\frac{1}{4}$ 1 down to base, they are sloped at $\frac{1}{2}$ 1 on the down-stream side and at $\frac{3}{4}$ 1 on the up stream face

The interesting old Bear Valley Dam is now submerged in the lake formed by the higher new dam of that name, which is matter for professional sadness Fig 15 reproduces its profile, together with stress-diagrams drawn by Wegmann and by Schuyler, respectively, which are based on

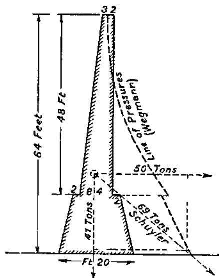


FIG 15

There have been two other high masonry dams in the world, which, though perfectly stable, have been treated as ignominiously as the Old Bear Valley Dam through the timidity of the local authorities These are

(2) The Zola Dam, near Marseilles, 120 feet high, 41 6 feet thick at base

(3) The Relleu Dam, in Catalonia, Spain, 100 feet high, 25 feet at base

When the storage of (2) proved insufficient the Marseillais built a con creted canal, 30 miles long, for supplementary water supply, rather than face the problem of building up the old dam higher And similarly, when the supply of the Relleu Dam, with base thickness only quarter of its height, needed augmentation, the Catalans, like the Californians, built a new and higher dam, entirely separate and down stream of the old one These were all horizontally arched dams, and from the point of view of engineering science it seems regrettable that one or other of them was not increased in height, without increase of base-thickness with a view to the extension of human knowledge

(EE) Par (10), Fig 2 —*The Pathfinder Dam*

This was built in 1909–10, in a granite canyon, only 80 feet wide at bottom, 180 feet at top and 190 feet deep Storage capacity 326,700 million gallons (U S A) for irrigation , catchment area, 10,500 square miles minimum flow, 400 cusecs , maximum, 13 000 cusecs , annual run-off $1\frac{1}{2}$ million acre-feet , length of crest, 425 feet , maximum height 210 feet , top thickness of dam, 10 feet , bottom thickness, 94 feet , up-stream face battered 25 per cent Calculations of design were based on the assumption that it would rely for stability on horizontal-arch action

of a flat deck demands considerable reinforcement, as well as frequent supports

(30) An example of a small work in type D is given in Fig 12 of the Schuylerville Weir, while an alternative design in type C has been prepared in Fig 13 and 13a The arrangement is similar to the last example (Fig 9), except that the piers are hollow No doubt a wider span than 20 feet would give better results

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Editor's Notes

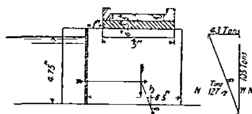
(E) Par (9), Fig 1 *Stresses in Arched Dams*

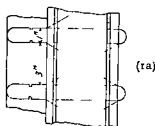
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* But the direct overfall will require a rock soil to resist the force of the falling water It is will not b so necessary in the Fig 12 type —Ed

prism of water 4 metres in length. The depth being 4·75 metres, the total pressure will be the area of the triangle of water pressure, which is $\frac{h^2}{2}$ multiplied by the length and by w the weight of a cubic metre of water or $\frac{wh^2}{2} \times l$. Taking w as equal to 1 ton, which it is very nearly, we have $r = \frac{(4{\cdot}75)^2}{2} \times 4 = 43{\cdot}1$ tons.

Now the cubical contents of one pier with superstructure lying to the right of the grooves is estimated at 60 cubic metres the weight of which, taking 1·75 for specific gravity of the brickwork, will be $1 \times 60 \times 1{\cdot}75 = 105$ tons. The portion of the pier up stream of the grooves has been omitted from consideration as it lies beyond the plane of pressure and is subjected to flotation. Strictly it should have been included in the count.





FIGS. 1, 1a.

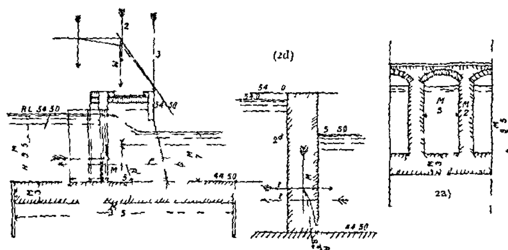
Fig. 1a is the force polygon. The incidence of the resultant R, on the base line, is almost exactly at the outer one-third division of the base. If it fell without to any extent, tension would be set up at the opposite toe, to avoid which the pier would have to be lengthened. The vertical line N in Fig. 1a measures the intensity of unit pressure on the masonry at the intersection of R and equals 127½ tons, which being situated at two thirds the length of the pier, the maximum at the toe will be $\frac{N}{b} \times 2 = \frac{127{\cdot}5}{6{\cdot}5} \times 2 = 40$ tons per square metre nearly (*vide* par. (8) Chap. II.) or as a square metre equals 10·75 feet the pressure per foot square will be $\frac{40}{10{\cdot}76}$ $= 3{\cdot}7$ tons. This is nearly double the safe pressure for ordinary soil. This calculation in the case of large spans with a greater head of water is very necessary, as not only may the pier be in tension at the heel but the pressure on foundations or floor may be excessive necessitating a lengthening of the pier down stream to increase its bearing surface.

(4) The thickness of piers is dependent on the weight carried and consequently is best expressed as some function of the span. The depth of water regulating the height of the piers is likewise a factor which must not be disregarded and further the allowable pressure on the foundation, though this latter can be arranged for by widening the pier foundation below the level of the floor base. The thickness may be generally taken as not under

$\sqrt{S}$ or to vary from $35S$ as a maximum to $25S$ as a minimum for spans from 10 to 25 feet The greater the span the less fractional proportion required for the pier Thus as in Fig 1 the width of the pier is $33S$ For a span of 24 feet under similar conditions the same fraction would give a width to the pier of 8 feet which is excessive 6 feet being a better proportion

(5) Until recently 25 feet used to be regarded as about the outside limit of workable vents for weir sluices and similar works whose functions are limited to partial regulation and in modern practice the least width of vent is hardly under 10 feet Since the invention of the Stoney pattern of live roller gear the economical limit of width has risen to 60 feet or even higher For canal heads subjected to a considerable head of water the thickness of the piers should be $4S$

(6) Of one quite modern example of regulating works viz the Assiut regulators a section is produced in Fig 2



Figs 2 a — Assiut Regulator

The piers in this work are 2 metres in width to spans of 5 metres giving a ratio of $4S$

(7) With regard to lateral pressure it is deemed that an extreme length of 6 metres is the outside exposed to this force The values of r and r_1 in this case will therefore be ₙ of the former values H remaining the same The lengths are projected on to an enlarged section of the pier in Fig 2d The final resultant falls just at the pier toe

(8) In the original design the piers were carried on circular wells of brickwork sunk in the sand The width of these was only 2 3 metres so that except in length the bearing surface on the sand was not diminished but neglecting the weight of the wells themselves and also the skin friction

the load on the actual base would not be less than 2 tons The modification of the original design actually executed is shown in Fig 2 mass concrete of one depth throughout of 3 metres enclosed in iron sheet piling having been substituted for a shallower floor and well foundations under the piers

(9) When a pier rests on a mass foundation of this description the load is gradually spread over a larger area and can be assumed with safety to splay out on each side in correspondence with that usually given to footings at a slope of ½ to 1

Thus as shown dotted in Figs 2 2a the bearing width at the base of the foundation will be the thickness of the pier plus that of the foundation in both cases 2 + 3 = 5 metres The same widening of effective bearing area may be conceived to occur at each end of the piers increasing the effective length from 13½ to 16½ metres

The vertical load on the foundation is 358 cubic metres to which must be added the contents below the floor or a piece 15 × 5 × 3 = 225 cubic metres submerged *i e* equivalent to 112 cubic metres (with $\rho = 2$) The total will then be 358 + 112 = 470 cubic metres = 940 tons

This load is distributed over a base of 16½ metres long and 5 metres wide consequently the pressure per square metre will be $\frac{940}{82\ 5} = 11\ 4$ tons and reduced to per square foot $\frac{11\ 4}{10\ 76} = 1\ 06$ tons In the case of the actual 5 metre spans the load will be less by a length of arching of 1 metre *i e* by some 10 or 15 cubic metres of masonry which would clearly make no appreciable difference

(10) We gather from this that a load of not more than 1 ton per square foot is admissible on a foundation of Nile silt On compact sand or hard clay a unit load of 2 tons is quite allowable On compact gravel blue clay or soft rock 4 tons is permissible

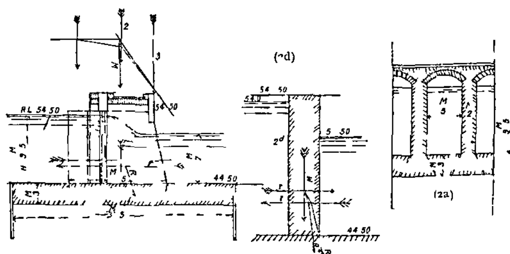
In the Nadrai Aqueduct Fig 3 Chap XI the pressure at the bottom of the well foundations was limited to 4 * tons and this on pure sand no allowance as usual having been made for skin friction which latter was ascertained by direct experiment to range from 6 to 1¾ cwt per square foot of surface On the Hawkesbury Bridge foundations also in sand 4½ tons was the limit pressure In some works on clay it amounts to over 8 tons per square foot Sand if protected from erosion is as good a foundation as any when of great depth if however the sand is only a thin layer over lying a clay bed the latter should be reached by protective curtain walls or sheet piling as otherwise the reduced frictional stability promoted by contact with a smooth hard surface might cause the sand layer to be washed out under hydrostatic pressure

* But see Editor s footnote to par (47) Chap I At a depth of 50 feet the resistance of the same soil to pressure would be about 9 tons per square foot greater than at ground surface —ED

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* But see Editor s footnote to par (47) Chap I At a depth of 50 feet the resistance of the same soil to pressure would be about 9 tons per square foot greater than at ground surface —Ed

(11) If the sand foundation is efficiently protected by curtain walls or sheet piling, no object is gained by deep foundations, and the same would apply to hard clay in cutting In the case of large spans where the floor is not sufficiently deep to afford the requisite support to the piers, the foundations of the piers must be carried lower A good example of how this is effected in mass foundations is given in Fig 3 of the pier bases of the Budki Superpassage

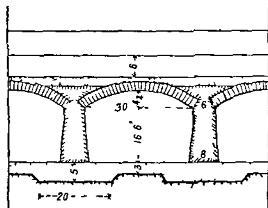


FIG 3

The required depth, as we have already seen, should be such that, added to the thickness of the pier, a base width is provided suitable to the limiting pressure on the subsoil

In regulating works, thick floors to resist the head of water are a necessity which is now thoroughly recognised, and the thickness given is often more than sufficient for pier foundations without anything supplementary in the way of special foundations being supplied underneath the piers

In the Budki Superpassage the pressure due to the weight of one span, plus that of 6 feet of water carried, is 2½ tons per square foot at the base of the piers and 1 45 tons on the ground The subsoil is hard, firm clay, 8 feet thick, overlying water-bearing sand Safe load, 2 tons per square foot

(12) The proportional thickness of piers for aqueducts varies considerably in different examples as below —

TABLE II *

Chap XI	Fig	Span.	Thickness	Ratio	$\sqrt{S}$	Radius (r)	$1 + 0.8\sqrt{r}$
Budki - -	8	30	6 ft	25	5 4	30	5 4
Thora Nala	1	30	5 ft	166S	5 4	20	4 6
Kerai	2	20	4 ft	2S	4 4	14	4 0
Gunnerum	4	40	6 ft	15S	6 3	33	5 7
Kali Nadi	3	60	7 ft	117	7 75	60	7 2

Aqueduct piers, like those of large span bridges, do not require to be vertically sided, as is necessarily the case in regulators, where draw-gates in grooves are used, and should always be constructed with either straight or curved battering sides, in order to increase the base width There is, however, a limit to the batter adopted, as the water-way is thereby diminished This subject is further treated in Chap XI

* The two last columns of this table have been added by the Editor of this edition for brickwork See Chap XI par (4)

Arches.

(13) The horizontal thrust of an arch is entirely dependent on the weight of that portion of the half arch and its load lying between the crown and the joint of rupture Whatever weight lies between the joint of rupture and the abutment does not affect the value of the thrust, as it really forms a projecting part of the abutment

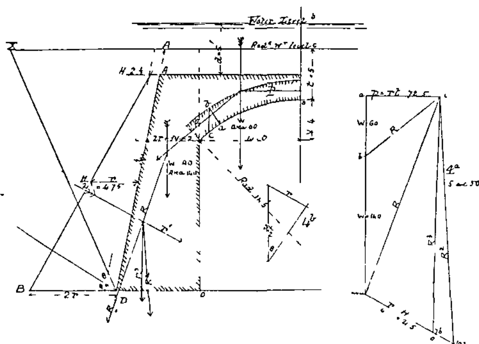
In every arch, of whatever profile, elliptical, segmental or semicircular, the point of rupture is that point where the internal line of pressure tends to go outside the middle third of the arch ring, and so produce tension in the arch Further, at this point, compared with others, the horizontal stress is at a maximum

The kind of arch most commonly used in Indian irrigation works is a circular arch forming a segment of 90 degrees or of 60 degrees, in which the point of rupture is at the springing The 90 degrees arch is that of least thrust, but the 60 degrees arch has less rise," which is sometimes advantageous

The thrust of an arch (P) loaded to a horizontal line above the crown is obtained by the following formula with great exactitude —

$$P = wrt, \qquad (1)$$

in which w is the unit weight of the material of the arch ring and filling, the latter reduced to an equivalent area of the material of the arch ring, r the radius of the intrados and t the vertical height of reduced terrain line above the intrados at the crown



FIGS 4 4a 4b

(14) An example illustrative of the arch thrust on abutments is given in Fig 4, which represents the segmental half arch of an aqueduct The

spandrel is built up to a horizontal line 6 inches above crown of arch. Above this lies 5 feet of water. This water area is reduced by dividing it by ρ the specific gravity of the material in the arch and in this case ρ is assumed as 2. The distance ac is thus the t in the formula and equals 5 feet. As the arch and abutment are considered to be of the same specific gravity, W which equals $w\rho$ is a common factor which can be eliminated in the graphical process. The horizontal thrust will then be represented by rt or by 14.5 $\times$ 5 = 72.5 and the weights of other parts by their areas. In Fig. 4a $ac = P = 72.5$ and the load line W or ab equals the area of the half arch viz. 60 the resultant being bc or R. In the profile Fig. 4 the position of P must be determined with reference not only to its own incidence in the arch ring at the crown but with reference to that of R in the arch ring in the springing. Both lines must fall within the middle third of the ring and that position of P which ensures the most axial incidence of both R and P is the correct one.

We now come to the composition of R with the weight of the abutment, which is W_1 on the load line representing the area of the trapezoid AO. The resultant R_1 falls just within the base.

The incidence should be somewhat more recessed within the toe hence the abutment requires thickening at the base.

Thickness of Abutments

15) The thickness of an abutment at the spring line of an arch must be such as to ensure against failure by sliding due to the horizontal thrust of the arch. A suitable width is obtained by use of the following empirical formula from Trautwine:—

$$t = .2r + .1V + 2 \qquad (2)$$

in which r is the radius of arch and V the versed sine.

In Fig. 4 the thickness is made in accordance with the formula being .2 × 14.5 + .1 × 4 + 2 = 5.3 feet. The rear slope is given by another formula—

Ratio of slope equals .025 horizontal to .51 vertical

$$\text{or } \frac{.025}{.5V}$$

The base width thus obtained will be just sufficient not counting in the assistance of the earth backing. Under normal conditions of loading these formulas would answer well but in the case in point the heavy load of water carried would require an increase in the thickness proportional to the depth carried. The increase adopted will be .2d or one fifth the depth of water, and the formula would become for aqueducts—

$$t = .2r + .1V + 2 + .2d \qquad (2a)$$

The increased thickness is shown by a dotted line in the profile Fig. 4.

In these calculations the abutment is assumed of the same specific gravity as that of the arch and loading.

(16) The effect of the earth backing is shown in Figs 4 4a and 4b In Fig 4b r or the half width of the triangle of earth pressure ADB is obtained by the process frequently employed in Chap I the specific gravity of the earth being taken as the same as that of the wall

The half width r_1 of the trapezoid of pressure $A\ B$ multiplied by its height gives its area and is equal to 4 75 × 21 5 = 102 By this quantity being set off in Fig 4a from the extremity of the load line in its proper direction we obtain R_3 on Figs 4a and 4

If the earth backing has three fourths the unit weight of the masonry the length of r_1 in Fig 4a will be three fourths less or 76 and the corresponding resultant is R in both figures In this and in the two preceding paragraphs the reader should note and distinguish between the letter (r) denoting radius of arch and the same letter denoting a force

Thickness of Arches

(17) There are several empirical formulas giving values of the crown thickness of arches of which the simplest and best is that in Molesworth s Pocket book of

$$t \quad n\sqrt{r} \text{ or } 4\sqrt{r} \tag{3}$$

n is a multiplier varying from 4 upwards

For ordinary bridges a value of 4 will be ample for the multiplier but this should be increased in the case of aqueducts up to 5 in proportion to depth of water carried (*vide* Chap XI) and the formula will then become

$$t = 5\sqrt{r} \tag{3a}$$

This is the thickness adopted in Fig 4

In large arches the thickness should increase proportionately with that of the thrust from the crown to the springing This increase can be obtained graphically as shown in Fig 4 Here ab is a radial equal to the thickness of the crown the vertical bc will then be the correct increased thickness at this point The span in this particular instance being small no increase is deemed necessary Here again the reader is requested to avoid the con fusion involved in the lettering a b c used in Figs 4 4a to denote different elements of the case illustrated

Buttresses

(18) Economy can sometimes be effected in the section of lofty abut ments or long retaining walls by the use of buttresses their effect on the stability of a retaining wall or abutment is obtained as follows —The wall should be considered as having a base width equal to the normal thickness at base plus the projection of the buttress but formed of two materials of different specific gravities the solid portion being of the actual specific gravity of the masonry and the area behind which is partly solid buttresses and partly open space should be considered as of a lighter specific gravity equivalent to that of a material occupying the whole of this rear area but weighing no more than the buttresses themselves

spandrel is built up to a horizontal line 6 inches above crown of arch. Above this lies 5 feet of water. This water area is reduced by dividing it by ρ the specific gravity of the material in the arch, and in this case ρ is assumed as 2. The distance ac is thus the t in the formula and equals 5 feet. As the arch and abutment are considered to be of the same specific gravity, W, which equals $u\rho$, is a common factor which can be eliminated in the graphical process. The horizontal thrust will then be represented by rt, or by 14.5 $\times$ 5 = 72.5, and the weights of other parts by their areas. In Fig. 4a $ac = P = 72.5$, and the load line W, or ab, equals the area of the half arch, viz., 60, the resultant being bc or R. In the profile Fig. 4, the position of P must be determined with reference not only to its own incidence in the arch ring at the crown, but with reference to that of R in the arch ring in the springing. Both lines must fall within the middle third of the ring, and that position of P which ensures the most axial incidence of both R and P is the correct one.

We now come to the composition of R with the weight of the abutment, which is W_1 on the load line representing the area of the trapezoid AO. The resultant R_1 falls just within the base.

The incidence should be somewhat more recessed within the toe, hence the abutment requires thickening at the base.

Thickness of Abutments.

(15) The thickness of an abutment at the spring line of an arch must be such as to ensure against failure by sliding due to the horizontal thrust of the arch. A suitable width is obtained by use of the following empirical formula from Trautwine:—

$$t = .2r + .1V + 2, \qquad (2)$$

in which r is the radius of arch and V the versed sine.

In Fig. 4 the thickness is made in accordance with the formula, being $.2 \times 14.5 + .1 \times 4 + 2 = 5.3$ feet. The rear slope is given by another formula—

Ratio of slope equals $.02S$ horizontal to $.5V$ vertical

$$\text{or } \frac{.02S}{.5V}$$

The base width thus obtained will be just sufficient, not counting in the assistance of the earth backing. Under normal conditions of loading these formulas would answer well, but in the case in point the heavy load of water carried would require an increase in the thickness proportional to the depth carried. The increase adopted will be $.2d$, or one-fifth the depth of water, and the formula would become for aqueducts—

$$t = .2r + .1V + 2 + .2d. \qquad (2a)$$

The increased thickness is shown by a dotted line in the profile Fig. 4.

In these calculations the abutment is assumed of the same specific gravity as that of the arch and loading.

(16) The effect of the earth backing is shown in Figs 4, 4a and 4b In Fig 4b, r, or the half width of the triangle of earth pressure ADB, is obtained by the process frequently employed in Chap I, the specific gravity of the earth being taken as the same as that of the wall

The half width r_1 of the trapezoid of pressure $A'B$, multiplied by its height gives its area, and is equal to 4 75 × 21 5 = 102 By this quantity being set off in Fig 4a from the extremity of the load line in its proper direction we obtain R_1 on Figs 4a and 4

If the earth backing has three-fourths the unit weight of the masonry, the length of r_1 in Fig 4a will be three-fourths less or 76, and the corresponding resultant is R_1 in both figures In this, and in the two preceding paragraphs, the reader should note and distinguish between the letter (r) denoting radius of arch, and the same letter denoting a force

Thickness of Arches.

(17) There are several empirical formulas giving values of the crown thickness of arches, of which the simplest and best is that in Molesworth s Pocket-book of

$$t = n\sqrt{r} \text{ or } 4\sqrt{r} \qquad (3)$$

n is a multiplier varying from 4 upwards

For ordinary bridges a value of 4 will be ample for the multiplier, but this should be increased in the case of aqueducts up to 5 in proportion to depth of water carried (*vide* Chap XI) and the formula will then become

$$t = 5\sqrt{r} \qquad (3a)$$

This is the thickness adopted in Fig 4

In large arches the thickness should increase proportionately with that of the thrust from the crown to the springing This increase can be obtained graphically as shown in Fig 4 Here ab is a radial equal to the thickness of the crown, the vertical bc will then be the correct increased thickness at this point The span in this particular instance being small no increase is deemed necessary Here, again, the reader is requested to avoid the confusion involved in the lettering a, b, c, used in Figs 4, 4a to denote different elements of the case illustrated

Buttresses.

(18) Economy can sometimes be effected in the section of lofty abutments, or long retaining walls, by the use of buttresses, their effect on the stability of a retaining wall or abutment is obtained as follows —The wall should be considered as having a base width equal to the normal thickness at base plus the projection of the buttress but formed of two materials of different specific gravities, the solid portion being of the actual specific gravity of the masonry, and the area behind, which is partly solid buttresses and partly open space, should be considered as of a lighter specific gravity equivalent to that of a material occupying the whole of this rear area, but weighing no more than the buttresses themselves

The specific gravity of this portion will then be $\rho \times \frac{a}{A}$ in which a is the area of the solid portion viz of the projecting buttresses and A that of the whole space between a line forming the ends of the buttresses and the back of the wall from which they project

Floors Thickness and Length

(19) *Firstly Of Submerged River Weirs on Sand*

The thickness of the floor is a matter dependent on the hydrostatic pressure to which the floor is subject owing to the porous nature of the substratum This is fully dealt with in Chap VI

The formula given in Chap VI is *

$$t = \tfrac{4}{3}\,\frac{H - h}{\rho - 1}$$

for submerged aprons or floor in which H is the maximum statical head and h the loss of head due to percolation up to the beginning of the floor

The width of submerged river weir aprons or floors on sand foundations is a function not only of the height of the permanent obstruction viz that of the masonry weir wall but also of the quality of the river sand represented by the so termed coefficient c and is found by the following formula also given in Chap VI

$$W = 4c\sqrt{\frac{H^a}{13}}$$

in which H^a is the height of the shutter crest For example if H^a be 12 feet and the river coefficient be of class 2 that is if $c = 12$ the width would be $4 \times 12\sqrt{93} = 48 \times 96 = 46$ feet nearly

20 *Secondly Canal Falls*

The thickness of the floor of canal falls without a sun water cushion should be $\sqrt{H + d}$ in which H is the height of the drop wall and d the depth of film i e the height of the upper reach above crest The width of floor should be $2(H + d)$ The same applies to escape falls

(21) *Thirdly Weir Sluices and other partial Regulators on sand*

The head to which a weir sluice is subjected being the same as that on the weir apron its floor could be made of the same length were it not for kinetic considerations the rush of water through the sluices particularly when the river is high being very great The decision is one for the judgment of the engineer on the facts of the case

Of partial regulators a good example is that of the Assiut regulators Although the water is over 30 feet in depth it is never required to be completely shut off so that the maximum hydrostatic pressure is due to a head of about 9 feet

In this case the thickness of the floor is 10 feet

* But see footnote to Chap VI par (13) —Ed

(22) *Fourthly Head Regulators or Intakes*

These works are sometimes subjected to great hydrostatical pressure Canal heads sometimes have to be entirely closed during the highest floods in the river to keep out excess of river silt The Egyptian canal heads are partial Head Regulators

If a Head Regulator is founded on good clay the floor thickness need not exceed $\sqrt{H_1}$ and the length be limited to 10 feet to 15 feet beyond the noses of the sluice piers

(23) The system of providing water cushions to canal falls by lowering the floor below the bed of the channel down stream involves considerable extra expense in that the height of the weir and retaining walls in fact all the parts of the structure have to be increased by the depth of the cushions In addition to this the difficulty of getting in the foundations in water bearing strata is greatly enhanced

Editor s Notes

(F) Par (19) —The formula for floor thickness on a sand foundation which the author recommends seems to err on the side of excessive caution as the reader will probably agree when he notices that it leads the author in Chap VII par (11) to reckon that the floor of the Narora Weir Sluices ought to be 10 feet thick whereas they are only half that thickness and yet have never exhibited any weakness Bearing in mind the fact that brick masonry or concrete when saturated with water is about 20 per cent heavier than when dry and bearing in mind also the transverse strength and power of resistance to shearing of a slab of such material we think that the factor (4/3) in the author s formula representing a factor of safety may well be omitted A vertical drop fall on a sandy river bed is in any case a bad design It has not been and is not likely to be adopted anywhere else in India except at Narora Our rough rule is to make a floor thickness of brick masonry or concrete equal to about half the probable hydrostatic head of blow up pressure

(FF) Par (23) —Strictly speaking such water cushions or cisterns below falls are useful chiefly in cases where the overfall is free or nearly so and where the water depth of the channel down stream is liable to be equal to or less than that of the channel up stream that is to say when the fall between water surfaces above and below the work is liable to be equal to or greater than the fall of the masonry work In the case of drowned falls the force of the passing water is exerted less vertically and more in the longitudinally forward direction in such cases therefore although a cistern may do some good in retarding velocity and force of current that purpose may be served better by lining the channel for some distance down stream by light but very rough masonry or pitching As a rough rule we would suggest that when the fall between water surfaces is less than one fourth of the water depth of the channel down stream no cistern need be designed in the masonry work

CHAPTER V

HYDRAULIC FORMULAS

(1) THE velocity of a jet issuing from an orifice under the head h is

$$c\sqrt{2gh} \qquad (1)$$

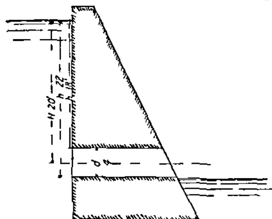


FIG 1

Fig 1 represents a sluice opening in a dam with a free outlet *i e* the level of the tail water is below the orifice In this case H is the mean head or depth of centre of sluice way below surface h_1 and h_2 are the depths to the top and bottom of the orifice l the width of the orifice whose depth is $h_2 - h_1$ or d Then the discharge will be

$$Q = cl\tfrac{2}{3}\sqrt{2g}\left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right) \qquad (2)$$

as $\sqrt{2g} = 8\ 025$ and $\frac{2}{3}\sqrt{2g} = 5\ 35$ the formula can be expressed as

$$Q = cl\ 5\ 35\ (h_2\sqrt{h_2} - h_1\sqrt{h_1}) \qquad (2a)$$

When H is large compared with d

$$Q = \text{(approximately) } cA\sqrt{2gH} \text{ or } cA \times 8\ 025\sqrt{H} \qquad (3)$$

A being the area of the orifice or $d \times l$

(2) In the example given in Fig 1 $H = 20$ $h_2 = 22$ $h_1 = 18$ $l = 4$ and $c = 66$

whence $Q = 66 \times 4 \times 5\ 35\ (22 \times 4\ 69 - 18 \times 4\ 242)$ cubic feet
$= 378\ 8$ cubic feet per second by formula (2a)

The discharge by formula (3) will be
$66 \times 16 \times 8\ 025 \times 4\ 472 = 379$ cubic feet per second

Hence in this case the shorter formula which always gives results somewhat in excess of (2) can well be used

The velocity of the current through the orifice will be according to (2a)

$$\frac{c \times 5\ 35\ (h_2\sqrt{h_2} - h_1\sqrt{h_1})}{d}$$

or according to (3) $c\sqrt{2gH}$ In most cases considering that the suitable value of the coefficient c is by no means known with anything like absolute

precision it would be a useless refinement to adopt the longer and more accurate formula (2) or (2a)

(3) Fig 2 represents a similar case in which the sluice or orifice is sub merged Here the head H is the difference in level between the head and tail water

The exact formula for the discharge through a submerged orifice is

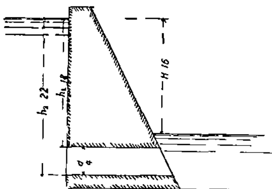
$$Q = cA\,5\,35\left\{(H\sqrt{H} - h_1\sqrt{h_1}) + 3\,\sqrt{H}(h_2 - H)\right\} \qquad (4)$$

In Fig 2 $H = 16$ h_1 and h being as before 18 and 22 feet

whence $Q = 66 \times 4 \times 5\,35\left\{(16 \times 4 - 18 \times 4\,243) + 3\,(4 \times 6)\right\}$

$= 14\,124\,(-12\,37 + 36) = 333\,75$ cubic feet

The approximate and more commonly used formula for discharge through a submerged orifice is the same as in (3) viz $Q = cA\sqrt{2gH}$ H however being the difference between head and tail water not the depth of the centre of the orifice below the surface In this instance $Q = 66 \times 16 \times 8\,025 \times 4 = 338\,98$ cubic feet the difference being 1 3 per cent in excess of the last However as already stated in the pre sent unsatisfactory state of hy draulic science the coefficients adopted are only approximate so that the shorter formula is quite good enough for practical purposes



FIG

In the above cases the same coefficient 66 has been used for both free and submerged orifices As will be seen later the coefficient is higher when the orifice is submerged

(4) In the two cases cited the question of the velocity of approach of the current if any has not been considered

Where the section of the orifice is not very small compared with that of the channel of supply this factor should be taken into account The velocity of approach can be expressed in terms of the head h expended in producing it as follows —

$h = \frac{V^2}{2g} = 0155\,V^2$ in this V is the mean velocity of the current in the channel of approach If observed surface velocity (v) only is known the mean velocity may be assumed to be $V = 7v$ or $8v$ A more accurate formula is given in par (13) The correct theoretical influence of the velocity

of approach is best obtained by modifying the coefficient so as to include it as follows —

$$c_1 = c\sqrt{1 + \frac{h}{H}} \qquad (5)$$

This obviates two more formulas to replace (2) and (3)

Where the depth of the orifice is small as regards the head of water, the following will give sufficiently accurate results viz —

$$Q = cA\ \sqrt{2g\,(H + h)} \qquad (6)$$

In this the head due to velocity of approach is simply added to the actual head of water which is not strictly accurate

(5) For example in Fig 1 supposing the mean velocity of the channel which supplies the sluice to be 3 feet per second then the modified coefficient will become $c_1 = 66\sqrt{1 \times \frac{h}{20}}$ Now h, or the head due to velocity of approach is as we have seen equal to $0155 \times 9 = 1395$ whence $c_1 = 66\sqrt{1\ 007} = 662$ The discharge will be increased in the first case of free outfall from 378 8 cubic feet to $378\ 8 \times \frac{662}{660}$ or to 379 95 cubic feet per second *

If formula (6) be used the discharge with free outfall will be $66 \times 16 \times 8\ 025\sqrt{20\ 1395} = 380\ 31$ With submerged orifice $H = 16$ and $Q = 66 \times 16 \times 8\ 025 \times \sqrt{16\ 1395} = 340\ 45$ cubic feet

These examples are useful as showing the discrepancy between the discharges worked out by the long or by the short formulas, and also showing that when H in either case is below $\frac{d}{5}$ (d being the depth of the orifice or sluice) the longer formulas (2) and (4) should be used instead of (3) and (6)

(6) In an orifice or sluice way contraction in the body of water passing through occurs on all four sides The section of the fluid is thus reduced below that of the opening and further the velocity of the water passing is decreased owing to the friction induced at the sides of the orifice below what it should be according to theory A reducing coefficient c has therefore to be used to include both these factors The exact determination of this coefficient is a matter of great importance For a circular orifice with thin sides as a hole cut in a thin plate the average coefficient obtained by experiment is about 62 As regards rectangular openings the value of c varies with the shape of the orifice as it has been found that an oblong hole gives a higher discharge than a square one of the same dimensions under the same head This proves that the end contractions of the vertical sides have a greater effect than those at the top and bottom of the orifice Hence a wide shallow orifice gives a higher discharge than a narrow and

* When velocity of approach is less than 4 feet per second the head due thereto is usually negl g ble —Ed

deep one The recognition of this fact should influence the design of sluice ways where it is desirable to reduce the head of water which must form above the sluice so as to overcome the frictional resistance of the current in its passage Cases illustrative of this point are noticed in Chap XI which deals with the design of masonry syphons

(7) The coefficient of discharge through a submerged orifice is always higher than that with a free outfall The same applies but in a more marked degree to cases of overfalls where there is no contraction on the upper surface Unfortunately no experiments have been made to ascertain the exact value of the coefficient under various values of H with which it naturally should vary In The Principles of Waterworks Engineering p 81 the coefficient for a submerged oblong orifice is stated to be experimentally 67 The ordinary coefficient applicable to small square openings is 62 In this particular case the opening is 2 feet × 6 inches *i e m* or the multiplier 1 05 whence for free outfall $c = 62 \times 1\ 05$ 65 Deduction from these data would make the coefficient for a submerged orifice under moderate head $\frac{62 \times 67}{65} = 639$ or say 64

In nearly all practical cases the orifice is submerged which accounts for the high range of many coefficients which have been obtained by testing the discharge through openings on a large scale on actual works and not on the minute dimensions of orifices and head from which the original coefficients were deduced

The rise of the coefficient due to submergence of an orifice shows that up to a certain point the rise in the back water has no influence on the discharge which is the same as if the orifice were clear There are no experiments which fix this limit but it can safely be assumed at ^1d

(8) The following are the coefficients applicable to different classes of orifices or sluices —

		C
1	Orifice in a thin plate or small iron sluice gate not at base of reservoir	= 62
	Same submerged	= 64
2	Sluices without side walls as tank sluices which open into wide culverts or discharge direct into the lower basin through a dam	= 62 to 66
3	Sluices situated at bottom of a reservoir as many tank sluices are	= 8
4	Narrow bridge openings bottom sluices in dam	= 8 to 9
5	Large sluice openings with side walls	= 9 to 94
6	Wide openings with bed level with that of the reservoir *i e* such as wide bridge openings with pointed piers	= 96

(9) From the foregoing it will be seen that the value of the coefficient is affected firstly by the size of the opening secondly by the provision of

of approach is best obtained by modifying the coefficient so as to include it, as follows —

$$c_1 = c\sqrt{1 + \frac{h}{H}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

This obviates two more formulas to replace (2) and (3)

Where the depth of the orifice is small as regards the head of water, the following will give sufficiently accurate results, viz —

$$Q = cA\ \sqrt{2g}\,(H + h) \quad . \quad . \quad (6)$$

In this the head due to velocity of approach is simply added to the actual head of water, which is not strictly accurate

(5) For example in Fig 1, supposing the mean velocity of the channel which supplies the sluice to be 3 feet per second, then the modified coefficient will become $c_1 = 66\sqrt{1 \times \frac{h}{20}}$ Now h, or the head due to velocity of approach is as we have seen, equal to 0155 × 9 = 1395, whence $c_1 = 66\sqrt{1\ 007} = 662$ The discharge will be increased in the first case of free outfall from 378 8 cubic feet to $378\ 8 \times \frac{662}{660}$ or to 379 95 cubic feet per second *

If formula (6) be used, the discharge with free outfall will be 66 × 16 × 8 025 $\sqrt{20\ 1395}$ = 380 31 With submerged orifice, $H = 16$ and $Q = 66 \times 16 \times 8\ 025 \times \sqrt{16\ 1395} = 340\ 45$ cubic feet

These examples are useful as showing the discrepancy between th discharges worked out by the long or by the short formulas, and also show that when H in either case is below $\frac{d}{5}$ (d being the depth of the orifice sluice), the longer formulas (2) and (4) should be used instead of (3) and

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* When velocity of approach is less than 4 feet per second the head due thereto is usually negligible —ED

except possibly in the case of a very small value of d, discharging over a thin plank A coefficient varying with the depth of film would probably be more strictly in accordance with experiments These experiments have, however, been undertaken on a very small scale, and a coefficient which may be truly applicable in cases of small depths of 2 feet or 3 feet passing over a narrow plank, the weir opening being likewise a small proportion of the normal width of the supply channel, will not prove so under actual practical conditions

The coefficient undoubtedly does decrease to some extent in inverse proportion with the depth of film, but on the other hand with a wide flat-topped masonry weir the increment is in the opposite sense, the fall in one case balancing to a greater or less degree the rise in the other, so that, pending further exhaustive experiments on a proper large scale the adoption of one value of c of 623 for all depths appears sufficiently accurate for practical purposes The letter Q signifies either the whole discharge or in the case of a weir, that per foot run, the absence of A or of l denoting which is meant

Besides which, this value is recognised generally as suitable, while that of Castel s, viz , 666 is not In the Madras Manual, the formula having a still lower value of c, viz 577, whence $Q = 3\ 1d\sqrt{d}$, is adopted for a free fall ' Per foot run " is sometimes designated *unit* discharge

(13) In Fig 3 the depth of film is shown as 4 feet The discharge per foot run of weir will then be $Q = 3\ 333 \times 4 \times 4 = 26\ 67$ cubic feet

In this, the velocity of approach, which may have to be considered, has been neglected

The easiest method is to modify the coefficient c, using formula (8) Let h equal height due to the mean velocity of approach or 0155 V^2 then this modified coefficient c_1 is obtained by use of the following formula —

$$c_1 = c \times \left\{ \left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}} \right\} \qquad (9)$$

For example supposing in the case already cited the mean velocity of the current to be $3\frac{1}{2}$ feet per second, then $h = 0155 \times (3\ 5)^2 = 19$ feet, and the multiplier of the coefficient with $d = 4$ and $h = 19$ is 1 062 and the discharge modified to include effect of velocity of approach will be $26\ 666 \times 1\ 062 = 28\ 32$ cubic feet per foot run The value (0155) is $\frac{1}{2g} = \frac{1}{64\ 4}$

(16) Fig 4 represents a submerged or drowned weir As in the case of a submerged orifice the head H is the difference in level of the head and tail water, or the afflux The depth of the film is termed as before, d, and the submerged portion of the film is $d - H$

The passing film thus consists of two portions, the upper having a free overfall and the lower being what can possibly be considered as a submerged orifice, but without any top contraction There can likewise be no bottom contraction or friction in the upper portion Some authorities, as Jackson,

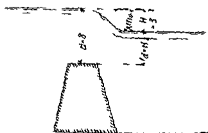
TABLE II.—DISCHARGES PER FOOT RUN OF FREE OVERFALLS

$$c = 623 \quad (\text{FORMULA})\ 3\ 333 d^{\frac{3}{2}} \left\{ \left(1 + \frac{h}{d}\right)^{\frac{3}{2}} - \left(\frac{h}{d}\right)^{\frac{3}{2}} \right\} \quad . \quad . \quad (10)$$

$$h = \cdot 0155 V^2$$

		MEAN VELOCITIES OF APPROACH IN FEET PER SECOND										
d	$3\ 333 d^{\frac{3}{2}}$	1	1½	2	2½	3	3½	4	4½	5	6	7
½	1 178	1 227	1 28-	1 353								
1	3 333	3 403	3 489	3 593	3 733							
1½	6 123	6 09	6 301	6 454	6 625	6 811						
2	9 427	9 531	9 653	9 804	10 002	10 65						
2¼	11 249	2 361	11 485	11 665	11 868	12 138	12 459	12 745	13 094	13 476		
2½	13 1 8	13 96	13 428	13 612	13 849	14 026	14 429	14 719				
3	17 319	17 457	17 6 3	17 804	18 064	18 375	18 722	19 085	19 518	19 963		
3½	1 8 4	21 9 7	22 129	22 318	22 631	22 959	23 352	23 766				
3¾	24 04	24 373	24 5 9	24 760	25 031	25 414	25 801	26 237	27 721	27 229		
4	26 664	26 824	26 984	27 224	27 544	27 917	28 317	28 770	29 240			
4½	31 8 7	31 976	32 166	32 42	32 739	33 121	33 566	34 076	34 617	35 221		
5	3 264	—	37 636	3 898	38 233	38 643	39 127	39 660	40 245	40 879		
5¼	40 094	—	40 494	40 735	41 136	41 537	42 018	42 579	42 982	43 822		
5½	42 991	—	43 422	43 636	44 066	44 452	44 969	45 528	46 129	47 075		
6	48 345	—	49 425	49 671	50 111	50 504	51 042	51 679	52 316	53 001	54 579	56 372
6½	55 2 4	—	55 676	56 007	56 394	56 816	57 389	57 996	58 713	59 431		
6¾	58 451	—	58 9 9	59 211	59 621	60 088	60 672	61 183	6 958	62 718		
7	6 28	—	62 160	62 531	62 901	63 395	64 012	64 691	65 370	66 111	67 864	70 006
7½	68 458	—	—	69 280	69 691	70 170	70 756	71 471	72 223	73·045	74 225	76 878
8	75 417	—	—	70 322	6 724	77 132	77 831	78 585	79 339	80 168	82·001	84 203
8½	81 597	—	—	82 505	83 836	84 414	85 075	85 818	86 651	87 470		
9	89 991	—	—	90 891	91 206	91 881	92 601	93 421	94 131	95 031	97 101	99 413
9½	3 534	—	—	98 472	98 960	99 545	100 23	101 01	101 88	102 86		
10	105 398	—	—	106 24	106 77	10 40	108 03	108 98	110 82	110 67	112 96	115 44
10½	113 40	—	—	114 19	114 76	115 41	116 01	117 14	118 05	118 96	121 11	123 72
11	12 63	—	—	122·60	—	123 82	—	125 5	—	127 37	129 70	132 58
12	135 425	—	—	139 44	140·00	140 68	141 44	142 33	143 27	144 38	146 80	149 47
13	56 08	—	—	—	—	158 42	—	160 14	—	162 33	165 51	167 32
13½	165 324	—	—	—	—	167 80	168 63	169 46	170·28	171 77	174 75	177 23
14	1 4 43	—	—	—	—	177 05	—	178 6	—	181 06	183 15	186 64
15	193 64	—	—	—	—	—	197 12	198 08	199 05	200 41	203 12	2 6 21
16	2 3 31	—	—	—	—	—	—	217 78	—	220 56	223 55	226 96
18	254 53	—	—	—	—	—	—	258 86	—	261 15	263 93	267 00
20	98 13	—	—	—	—	—	—	303 59	304 66	306 03	309 30	313 06

calculate the discharge of each portion with two separate values of c, the upper by formula (8), the lower by formulas (2) or (3) Jackson uses the coefficient 66 for the upper and 62 for the lower The use of so low a coefficient as 62 for the lower orifice portion is clearly quite indefensible, it undoubtedly should vary from 7 to 9 or more * In the case of an ordinary orifice, the coefficient 84 given in par (8) would seem suitable If, however, a high coefficient be used, the discharge will, in many cases exceed that of a free overfall, which is clearly impossible—hence the reason of Jackson's adoption of the low coefficient


FIG 4

* The Madras formula which has been well tested makes the coefficient c_1 for the "free" portion of the overfall = 577 and c_2 for the drowned portion = 80 The formula is $Q = \frac{2}{3} c_1 \sqrt{2g} \left\{ (H + H_a)^{\frac{3}{2}} - H_a^{\frac{3}{2}} \right\} + \left\{ c_2 \sqrt{2g}\, (H + H_a)^{\frac{1}{2}} \times x \right\}$ where x is depth of tail-water over crest and H_a is head due to velocity of approach Perhaps it would be sounder to say c_1 = 577 to 667 according to shape of mouthpiece and c_2 = 75 to 85 according to depth of drowning —ED

But further, cases do occur where, by using this method and these coefficients, the resulting discharge will exceed that of a free overfall The actual conditions of the two portions of the film are so dissimilar from the hypothetical, that this method of estimating the discharge cannot be satisfactory, and the only alternative is to adopt one coefficient for the whole film, neglecting any imaginary horizontal division in the same

(18) The following method, first instituted by Herschel, an American hydraulician, from the experiments of Francis and of Fteley and Stearns, appears to give satisfactory results within certain limits, and has the great merit of simplicity

The formula for free discharge per foot run, or $Q = 3\ 33 d^{\frac{3}{2}}$ *is modified as* follows —

$$Q = 3\ 33\ (nd)^{\frac{3}{2}} \tag{8a}$$

The multiplier n varies in the proportion of d $(d - H)$ The results are liable to a probable error of about one unit in the second decimal place when $(d - H)$ is less than $2H$ and to greater errors as the table progresses, values of n under 7 being in particular uncertain

The following table of values of n has been derived from Merriman s " Hydraulics " —

TABLE III —FACTORS (n) FOR SUBMERGED WEIRS

In formula (8a) for discharge of submerged weirs per foot run, $Q = 3\ 33\ (nd)^{\frac{3}{2}}$

$\frac{d-H}{d}$	n	$\frac{d-H}{d}$	n	$\frac{d-H}{d}$	n	$\frac{d-H}{d}$	n
0 00	1 000	0 18	0 989	0 38	0 935	0 58	0 856
01	1 004	20	985	40	929	60	846
02	1 006	22	980	42	922	62	836
04	1 007	24	975	44	915	64	824
06	1 007	26	970	46	908	66	813
08	1 006	28	964	48	900	70	787
10	1 005	30	959	50	892	75	750
12	1 002	32	953	52	884	80	703
14	998	34	947	54	875	90	574
16	994	36	941	56	866	1 00	0 000

(19) The question of velocities * of approach and their values appears not to have received attention in works on hydraulics but is passed over in discreet silence consequently the only way to deal with the question is to use the same values as appear in *Table II* multiplying up *all* the columns by the new factor $n^{\frac{3}{2}}$ This has been done in the following Tables which give a sufficiently approximate value to work on, until reliable coefficients are made available by means of experiments

* See Editor s Notes at end of this Chapter —ED

TABLE IV. (SERIES II)—DISCHARGES PER FOOT RUN OF SUBMERGED FALLS

$H = 1$

d	$\frac{d-H}{d}$	$n^{\frac{1}{2}}$	Mean Velocities of Approach							
			0	1	2	3	4	5	6	7
1½	3	94	5 7	5 8	6 1	6 4				
2	5	84	7 9	8 0	8 2	8 6				
2½	6	79	10 2	10 3	10 6	10 9	11 4			
3	66	73	12 7	12 8	13 0	13 5	14 0			
3½	7	70	15 3	15 4	15 6	16 0	16 6			
4	75	65	17 3	17 4	17 7	18 1	18 6	19 0		
4½	77	63	20 0	20 1	20 4	20 8	21 5	22 1		
5	8	59	22 0	—	22 3	22 8	23 4	24 1		
5½	82	56	24 1	—	24 4	24 9	25 5	26 3		
6	83	54	26 4	—	26 8	27 3	27 9	28 6	29 5	
7	85	51	31 5	—	31 9	32 3	33 0	33 7	34 6	
8	87	48	36 2	—	36 6	37 0	37 7	38 5	34 6	

TABLE IV. (SERIES III)—DISCHARGE PER FOOT RUN OF SUBMERGED FALLS

$H = 1\frac{1}{2}$

d	$\frac{d-H}{d}$	$n^{\frac{1}{2}}$	Mean Velocities of Approach							
			0	1	2	3	4	5	6	7
2½	4	93	12 2	12 4	12 6	13 0	13 6			
3	5	84	14 5	14 8	15 0	15 4	16 0			
3½	57	81	17 6	17 8	18 1	18 6	19 2			
4	62	76	20 2	20 4	20 7	21 2	21 6			
4	67	72	22 9	23 1	23 3	23 8	24 5	25 3		
5	7	70	26 1	—	26 5	27 0	27 8	28 1		
5½	73	67	28 8	—	29 2	29 8	30 5	31 5		
6	75	65	31 8	—	32 3	32 8	33 6	34 6		
6½	77	63	30 9	—	31 3	31 8	3[illegible] 5	33 4		
7	79	59	36 4	—	36 9	37 4	38 2	39 0		
7½	8	58	39 7	—	40 2	40 7	41 4	42 4	43 0	44 6
8	81	57	43 0	—	43 5	43 9	44 8	45 6	46 8	48 0
8½	82	56	45 7	—	46 2	47 3	48 0	48 9		
9	83	54	48 6	—	49 0	49 6	50 4	51 3	52 3	53 7
9½	84	53	51 7	—	52 2	52 7	53 5	54 5		
10	85	51	53 7	—	54 2	54 8	55 6	56 4	57 1	58 9
11	86	49	59 6	—	60 0	60 7	61 3	62 5	63 6	65 0
12	87	48	66 4	—	66 9	67 5	68 3	69 3	70 5	71 7

TABLE IV (SERIES IV)—DISCHARGE PER FOOT RUN OF SUBMERGED FALLS

$H = 2$

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach								
			0	1	2	3	4	5	6	7	8
2½	2	98	12.9	13.0	13.3	13.7					
3	33	92	15.9	16.1	16.5	16.9	17.6				
3½	4	89	19.5	19.7	20.0	20.5	21.3				
4	5	84	22.3	22.5	22.8	23.4	24.2				
4½	55	81	25.8	25.9	26.2	26.8	27.5	28.5			
5	6	78	29.0	—	29.5	30.1	31.0	31.8			
5½	64	75	32.1	—	32.7	33.3	34.1	36.1			
6	67	72	35.3	—	35.8	36.4	37.2	38.2			
6½	69	71	39.2	—	39.8	40.3	41.2	42.2			
7	71	68	42.0	—	42.5	43.1	44.0	45.0	46.1	47.6	
7½	73	67	45.8	—	46.4	47.0	47.9	48.9	49.7	51.5	
8	75	65	48.0	—	49.6	50.1	51.1	52.1	53.3	54.7	
8½	76	63	51.4	—	52.0	53.2	54.1	55.1			
9	78	61	54.9	—	55.4	56.1	57.0	58.0	59.2	62.6	
9½	79	60	58.5	—	59.1	59.7	60.6	61.7			
10	80	59	62.2	—	62.7	63.4	64.3	65.3	66.7	68.0	
11	82	56	68.1	—	68.6	69.3	70.3	71.5	72.7	74.2	
12	83	54	74.7	—	75.3	75.9	76.8	78.0	79.2	81.0	

TABLE IV (SERIES V)

$H = 2\frac{1}{2}$

d	$\frac{d-H}{d}$	$n^{\frac{3}{2}}$	Mean Velocities of Approach							
			0	2	3	4	5	6	7	8
3½	28	95	20.7	21.2	21.8	22.6				
4	37	91	24.4	24.7	25.4	26.2				
4½	44	87	27.7	28.2	28.8	29.6	30.6			
5	5	84	31.3	31.9	32.5	33.3	34.3			
5½	53	82	35.2	35.8	36.4	37.3	38.6			
6	58	79	38.7	39.2	39.9	40.8	41.9	42.5		
6½	61	77	42.5	43.1	43.7	44.7	45.8			
7	64	75	46.3	46.9	47.5	48.5	49.6	50.9	52.50	
7½	66	73	50.0	50.6	51.2	52.2	53.3	54.2	56.1	
8	69	71	53.5	54.2	54.8	55.7	56.9	58.2	59.8	
8½	7	70	57.1	57.7	59.1	60.0	61.2			
9	72	68	61.2	61.8	62.5	63.4	64.6	66.0	67.6	
9½	73	67	65.4	65.9	66.7	67.6	68.9			
10	75	65	68.5	69.0	69.8	70.8	71.9	73.4	75.0	
11	77	63	76.6	77.2	78.0	79.1	80.3	81.7	83.5	
12	80	59	81.7	82.3	83.0	84.0	85.2	86.6	88.2	
13	81	57	89.0	—	90.3	91.8	92.5	94.3	95.4	
14	82	56	97.6	—	99.0	100.0	101.4	102.6	104.5	
15	83	54	104.6	—	—	107.0	108.2	109.7	111.35	
16	84	53	113.0	—	—	115.4	116.9	118.5	120.3	
18	85	51	130.3	—	—	132.0	133.2	134.7	136.2	
20	87	48	142.9	—	—	145.7	146.9	148.5	150.3	

TABLE IV. (SERIES VI)

$H = 3$

d	$\frac{d-H}{d}$	$n^{\frac{2}{3}}$	Mean Velocities of Approach						
			0	2	3	4	5	6	7
4	25	96	25 6	26 1	26 8	27 6			
4½	33	93	29 6	30 1	30 8	31 7			
5	4	89	33 2	33 7	34 4	35 3	36 4		
5½	45	87	34 8	35 4	36 1	37 0	38 1		
6	5	84	41 1	41 7	42 4	43 4	44 5	45 8	
6½	54	82	45 3	45 9	46 6	47 5	48 7		
7	57	80	49 4	50 0	50 7	51 7	52 8	54 3	56 0
7½	60	78	53 4	54 0	54 7	55 7	56 9	57 9	59 0
8	62	76	57 3	58 0	58 6	59 7	60 9	62 3	64 0
8½	64	75	61 2	61 9	63 4	64 4	65 6		
9	66	73	65 7	66 3	67 0	68 1	69 4	70 9	72 6
9½	68	71	69 3	69 9	70 7	71 7	73 0		
10	7	70	73 6	74 3	75 2	76 3	77 5	79 0	80 7
10½	72	68	77 1	77 6	78 5	79 6	80 7	82 3	84 1
11	73	67	81 5	81 9	82 9	84 2	85 4	87 3	88 9
12	75	65	90 0	90 6	91 4	92 5	93 9	95 4	97 1
13	77	63	98 3	—	99 8	100 8	102 2	103 9	105 4
14	79	60	104 6	—	106 1	107 1	108 6	109 9	112 0
15	80	59	114 1	—	—	116 9	118 1	119 7	121 8
16	81	57	121 6	—	—	124 1	125 7	127 4	129 4
18	83	54	137 4	—	—	139 8	141 0	142 5	144 2
20	85	51	152 0	—	—	154 7	155 4	157 8	159 6

TABLE IV. (SERIES VIII)

$H = 4$

d	$\frac{d-H}{d}$	$n^{\frac{2}{3}}$	Mean Velocities of Approach						
			0	2	3	4	5	6	7
6	33	93	45 6	46 3	47 0	48 1	49 7	50 8	
6½	38	90	49 7	50 4	51 2	52 2	53 5		
7	43	88	54 3	55 0	55 8	56 9	58 2	59 7	61 6
7½	47	86	58 9	59 6	60 3	61 5	62 8	63 8	66 1
8	50	84	63 4	64 1	64 8	66 0	67 3	68 9	70 7
8½	53	82	67 0	67 6	69 2	70 4	71 7		
9	55	81	72 9	73 7	74 4	75 6	77 0	78 6	80 5
9½	58	79	77 1	77 8	78 6	79 8	81 3		
10	60	78	82 2	82 9	83 7	85 0	86 3	88 1	89 6
11	64	75	91 2	92 0	92 9	94 1	95 5	97 3	99 4
12	67	72	99 7	100 3	101 3	102 6	103 9	105 7	107 6
13	69	71	110 8	—	112 5	113 7	115 3	117 5	118 8
14	71	69	120 4	—	122 2	123 3	124 9	126 4	128 8
15	73	67	129 7	—	—	132 7	134 2	136 1	138 2
16	75	65	138 6	—	—	141 6	143 4	145 3	147 5
18	78	61	155 2	—	—	157 9	159 3	161 0	162 9
20	80	59	175 9	—	—	179 1	180 6	182 5	184 7

(20) The mean velocity of approach V_a which we have been considering is naturally some function of that of the normal river current. Now the discharge $Q = AV = A_1 V_a$ in which expression A_1 is the area of the water section at or near the weir or obstruction. In deep reservoirs which do not silt up to any great extent V_a is less than V. A proper appraisement of the value of A_1 is essential to the correct valuation of V_a.

A point to be noted is that the mean velocity of approach of a large wide river varies in different parts of the water section and in such cases the river and the weir should be divided into parts each with its own velocity of approach and the multiplier varied accordingly.*

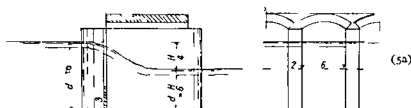
(21) In the above treatment of weirs we have dealt with open weirs presumably across rivers; in canals, however, the superiority of the notch type of weirs which enables the water in the upper reach to be held up to normal level at whatever depth it may be renders its adoption suitable.

In a notch fall the triangular or rather trapezoidal openings are built with a converging and diverging approach similar to the so termed adjutage attached to an orifice. This method of construction greatly increases the value of the coefficient of discharge through the notch so that when thus constructed the modification due to end contraction need not be used as it would have to be were the sides of the notch left square, instead of which the coefficient should be slightly reduced.

It is considered that the coefficient for notch falls should be reduced † from the 623 of Francis formula to one of about 56. This will make with notch falls $Q = 3d$ per foot run. This value is just 10 per cent below that for ordinary falls as calculated from Francis coefficient, consequently new tables are not required.

(23) We will now proceed to examine the case of a fall through the openings of a head regulating bridge, escape head or undersluices.

Figs 5 and 5a are views of a work of this description; the surface of water being free the case apparently resolves itself into the simple one of a



Figs 5, 5a

submerged overfall to which formula (8a) is applicable. The conditions however present some differences to those of a fall over a weir wall. The presence of the bottom floor diminishes the friction at the base of the current,

* In practice it will usually be difficult to apply this process with satisfactory accuracy.—Ed.

† On Punjab canals the coefficient used is commonly [illegible] for canals and [illegible] for distributaries. These [illegible] cover allowance for velocity of approach.—Ed.

and consequently in such cases as noticed in par (9) the coefficient always rises in value Against this rise is the diminution in effective width due to the piers As seen in par (11), the reduction in the effective length of the water section would be ιnd, or $2 \times 10 = 2$ feet This, however, applies only to the square and raised flanks of an ordinary weir wall, and not to piers provided with well shaped cutwaters as should invariably be adopted In the latter case the effect of the cutwaters is to modify in a great measure the contraction alluded to

(24) With regard to the coefficient, the ordinary one of 623 would clearly be inapplicable In the case of orifices, proper reference to the list in par (8) shows that the rise of the coefficient in the case of bottom sluices is roughly about 2 points *i e*, from about 7 to 9 This would apply to the undercurrent portion of the fall only so that it is considered that the coefficient should be raised from 623 to 66 (the Castel coefficient) The discharge* under such conditions would be that given by Table IV increased by $\frac{66}{62}$ or by $\frac{33}{31}$ For example in this case $H = 4$ $d = 10$, in Series VIII the unit discharge due to this combination is 82 2 cubic feet which, increased by $\frac{33}{31}$, will become 87 5 * cubic feet per second and in a 6-foot bay, 6×87 5, or 525 cubic feet per second

(25) If the arch were depressed below the surface of the water up stream as is often the case in head and undersluices the surface would not be free and contractio [illegible]
cause the litte [illegible]
be applicable
In this case 90 would probably apply

If a bottom gate or baulk were left in the grooves in Fig 5 the discharge would be considerably modified Supposing this obstruction to be 3 feet deep then on the assumption that H is of the same value viz, 4 feet, d will be reduced to 7 feet and the coefficient falls back to 623

As a matter of fact H would rise till equilibrium was produced somehow, a matter very difficult to estimate with any precision unless the whole river supply went through the sluices

The depth of the escape channel *i e*, of the tail water or $d - H$, depends on the slope of the channel and on its sectional area, *i e*, its discharging capacity and has been dealt with in another place

(26) In a manner similar to the case of notch falls, a reduction in discharge per foot run can be effected in order to make allowance for the shape

* According to the Madras formula the discharge per foot run neglecting velocity of approach would be

(*a*) For the free portion $Q_1 = \frac{2}{3}c_1\sqrt{2g}\,(H)^{\frac{3}{2}} = 3\ 1H^{\frac{3}{2}} = 24\ 8$ (where $c_1 = 577$)

(*b*) For the drowned portion $Q_2 = c_2(d - H)H^{\frac{1}{2}} = 8 \times 6 \times 4^{\frac{1}{2}} \times 8\ 025 = 76\ 8$
Whence $Q = Q_1 + Q_2 = 24\ 8 + 76\ 8 = 101\ 6$ per foot run —ED

of the weir crest If the latter is broad crested like that of type C in Chap VI., a coefficient of ·577 is often adopted in lieu of 623

Discharge of Channels

(27) The formula for discharge of channels is Chezy's with Kutter's coefficient, viz ,

$$Q = AV = AC\sqrt{RS} \qquad (13)$$

in which R is the hydraulic mean depth, or $\frac{\text{area of section}}{\text{wetted perimeter}}$, and S is the sine of the slope of water surface which can be assumed as parallel to the average slope of the bed of channel The coefficient C is obtainable from Kutter s formula, which has superseded all previous ones This formula expressed in feet is

$$C = \frac{41\ 6 + \frac{1\ 811}{n} + \frac{00281}{S}}{1 + \left(41\ 6 + \frac{00281}{S}\right)\frac{n}{\sqrt{R}}} \qquad (14)$$

in which n, termed the coefficient of rugosity, varies with the nature of the channel and its degree of smoothness and uniformity of section The values of n for earthen channels are as follows —

For firm soil, of uniform section	$n =$ 020
For channels in earth in good condition	$n =$ 0225
For channels in earth in average condition	$n =$ 025
For channels in earth below average condition	$n =$ 0275
For channels in earth in defective condition, or rivers	$n =$ 030
For very irregular channels	$n =$ 035 to 0 060 or even higher

In calculations for canal discharges, n is given a value either of 025 or 0225 in Punjab Canals practice, but values above 0 027 often obtain in practice, in head reaches of large canals, much silted, or scoured

(28) This formula (14) being somewhat cumbersome, tables are sometimes made use of in its application Jackson s Hydraulic Manual ' and Higham's and Colonel Moore's tables are useful It is sometimes convenient to substitute for S, the sine of the inclination of the water surface, S in 1,000, termed $S°$ The formula for velocity can then be expressed as $V = c\sqrt{10RS°}$.

Thus if $R = 4$ and $S°$ is 2 per 1,000, v will equal $2c\sqrt{2} = 2\ 83c$ cubic feet per second

For channels other than earth the values of n are as follows —

Well planed planks	$n = 009$
Cement-plaster, or enamelled pipes	$n = 010$
Cement-and sand plaster	$n = 011$
Unplaned timber	$n = 012$
Ashlar and brickwork cast and wrought iron	$n = 013$
Rubble masonry	$n = 017$

The value of n, the coefficient of rugosity has a great effect on the discharge so the selection of a suitable value in the case of river discharges is very essential The coefficient in the case of natural channels will vary usually from $n = 0250$ to 0350 for perennial streams and up to 0 060, or, higher for torrents

(29) The published tables by Colonel Moore, R E (Batsford), entitled New Tables for the Complete Solution of Ganguillet s and Kutter's Formula are of great use in finding the velocity, though for the reverse processes of finding S corresponding to certain values of R and of V they are not so suitable as Jackson s or Higham s In this work Chezy's form of equation viz $V = 100\, c \sqrt{RS}$ is not used but Kutter s long formula, which gives the velocity direct, not the derived coefficient applicable to Chezy s formula given in par (27) is worked out

The long formula as modified by Colonel Moore is

$$V = \frac{\frac{l}{n} + \left(a + \frac{m}{S}\right)}{\sqrt{R} + \left(a + \frac{m}{S}\right) n} \quad R\sqrt{S}$$

This is abbreviated into $V = \frac{NR}{\sqrt{R} + D}$

where $$N = \left(\frac{l}{n} + \left(a + \frac{m}{S}\right)\right) \sqrt{S}$$

and $$D = \left(a + \frac{m}{S}\right) n$$

The tables give the values of N log N and of D for every possible value of S or surface inclination from 1 over 1 to 1 over 20 000

The value of R has to be calculated or found in the tables given it being equal to $\frac{A}{W\,P}$ or the area divided by the wetted perimeter Thus by putting a few figures together the value of V is obtained A large diagram is also annexed in which the value of c applicable to Chezy s formula can be obtained These tables form a valuable labour-saving help in the manipulation of Kutter s formula

(30) Methods of calculating flood discharges of rivers

As already noted the discharge of any channel is $A \times V$ in which A is the area of the water section and V the mean velocity of the current The mean velocity can be deduced by actual observation of the surface velocity in feet per second and the latter multiplied by a proper coefficient which varies from 65 to 8 gives the mean velocity (*vide* par (31))

In a wide stream several surface velocities can be observed the water section being divided into any convenient number of parts and the floats are caused to pass through the centre of each of these divisions as near as possible then the discharge will be the sum of these areas each multiplied by the respective observed surface velocity reduced to mean velocity

A more accurate method is by means of vertical rod floats of various lengths These can be either tin tubes weighted at the bottom so as to float nearly immersed or else wooden rods with a piece of lead gas piping twisted round the bottom These velocity rods should float just clear of the bed of the stream If the latter is very uneven as is often the case in rivers the system will not work The rods give the actual mean velocity at once and consequently furnish more accurate results than reduced surface observations

Another method of obtaining actual mean velocity observations is by the use of current meters The meter is immersed at $\frac{3}{5}$ the depth of the water in the middle of each division of the water area section and the velocity is recorded automatically by an electric current the meter being connected with a battery

(31) To determine the mean velocity in a cross section of a current where the maximum surface velocity is known the following formula by Prof von Wagner gives reliable results —

$$V = 705 \times 003 v^2 \qquad (15)$$

In which V is the mean and v the surface velocity

Thus supposing the observed maximum surface velocity of a section to be 3 feet per second the mean velocity V will be $705 \times 3 + 003 \times 9 = 2\ 115 + 027 = 2\ 142$ feet per second *i e* a little over 7 V Other authorities as Jackson give 8 as the coefficient In canal and river discharges in India 7 or 75 are the coefficients usually employed so that the ratio given in formula (15) can well be adopted

(34) It often happens that actual velocity observations cannot be taken during a freshet and there is no time available for putting up self recording white gauge posts which will indicate the flood surface by the discoloration due to muddy water passing In such cases the slope of the water surface being unknown it can be assumed as parallel to the average bed slope for which purpose a longitudinal section of the bed for at least a mile in length should be levelled and plotted and at least three cross sections taken of the channel showing the maximum flood level which can be approximately obtained by observation of floating detritus deposited on

For channels other than earth the values of n are as follows —

Well planed planks	$n = 009$
Cement plaster or enamelled pipes	$n = 010$
Cement and sand plaster	$n = 011$
Unplaned timber	$n = 012$
Ashlar and brickwork cast and wrought iron	$n = 013$
Rubble masonry	$n = 017$

The value of n the coefficient of rugosity has a great effect on the discharge so the selection of a suitable value in the case of river discharges is very essential The coefficient in the case of natural channels will vary usually from $n = 0250$ to 0350 for perennial streams and up to 0 060 or higher for torrents

(29) The published tables by Colonel Moore R E (Batsford) entitled New Tables for the Complete Solution of Ganguillet s and Kutter s Formula are of great use in finding the velocity though for the reverse processes of finding S corresponding to certain values of R and of V they are not so suitable as Jackson s or Higham s In this work Chezy s form of equation viz $V = 100 c \sqrt{RS}$ is not used but Kutter s long formula which gives the velocity direct not the derived coefficient applicable to Chezy s formula given in par (27) is worked out

The long formula as modified by Colonel Moore is

$$V = \frac{\frac{l}{n} + \left(a + \frac{m}{S}\right)}{\sqrt{R} + \left(a + \frac{m}{S}\right) n} R\sqrt{S}$$

This is abbreviated into $V = \dfrac{NR}{\sqrt{R} + D}$

where $$N = \left\{\frac{l}{n} + \left(a + \frac{m}{S}\right)\right\} \sqrt{S}$$

and $$D = \left(a + \frac{m}{S}\right) n$$

The tables give the values of N log N and of D for every possible value of S or surface inclination from 1 over 1 to 1 over 20 000

The value of R has to be calculated or found in the tables given it being equal to $\frac{A}{W P}$ or the area divided by the wetted perimeter Thus by putting a few figures together the value of V is obtained A large diagram is also annexed in which the value of c applicable to Chezy s formula can be obtained These tables form a valuable labour saving help in the manipulation of Kutter s formula

(30) Methods of calculating flood discharges of rivers

As already noted, the discharge of any channel is $A \times V$, in which A is the area of the water section and V the mean velocity of the current The mean velocity can be deduced by actual observation of the surface velocity in feet per second, and the latter multiplied by a proper coefficient, which varies from 65 to 8, gives the mean velocity (*vide* par (31))

In a wide stream several surface velocities can be observed, the water section being divided into any convenient number of parts, and the floats are caused to pass through the centre of each of these divisions as near as possible, then the discharge will be the sum of these areas, each multiplied by the respective observed surface velocity, reduced to mean velocity

A more accurate method is by means of vertical rod floats of various lengths These can be either tin tubes weighted at the bottom so as to float nearly immersed, or else wooden rods with a piece of lead gas piping twisted round the bottom These velocity rods should float just clear of the bed of the stream If the latter is very uneven as is often the case in rivers the system will not work The rods give the actual mean velocity at once and consequently furnish more accurate results than reduced surface observations

Another method of obtaining actual mean velocity observations is by the use of current meters The meter is immersed at $\frac{1}{2}$ the depth of the water in the middle of each division of the water area section and the velocity is recorded automatically by an electric current, the meter being connected with a battery

(31) To determine the mean velocity in a cross section of a current where the maximum surface velocity is known, the following formula by Prof von Wagner, gives reliable results —

$$V = 705 \times 003 v^2 \qquad (15)$$

In which V is the mean and v the surface velocity

Thus supposing the observed maximum surface velocity of a section to be 3 feet per second, the mean velocity V will be $705 \times 3 + 003 \times 9 = 2\ 115 + 027 = 2\ 142$ feet per second *i e*, a little over 7 I Other authorities, as Jackson, give 8 as the coefficient In canal and river discharges in India 7 or 75 are the coefficients usually employed, so that the ratio given in formula (15) can well be adopted

(34) It often happens that actual velocity observations cannot be taken during a freshet and there is no time available for putting up self recording white gauge posts which will indicate the flood surface by the discoloration due to muddy water passing In such cases the slope of the water surface being unknown, it can be assumed as parallel to the average bed slope, for which purpose a longitudinal section of the bed for at least a mile in length should be levelled and plotted, and at least three cross sections taken of the channel showing the maximum flood level, which can be approximately obtained by observation of floating detritus deposited on

the bank or on bushes and by inquiry These three cross sections taken at different places should be plotted one over the other and from this an average representative cross section should be drawn Having obtained S *i e* the sine of the bed slope by levelling A and WP are measured from the section and thus R is obtained The probable value of n the coefficient of rugosity has to be fixed with reference to the natural state of the channel and by application of formula (13) viz $Q = 100\, Ac\sqrt{RS}$, the discharge is obtained The value of the expression $100\sqrt{RS}$ can be obtained from Jackson s Hydraulic Manual or other tables in which Kutter s formula is worked out for English feet

(36) The usual procedure of taking velocity discharges is as follows — A convenient reach being selected poles are erected in pairs at the starting and finishing points on either side of the channel their direction being at right angles to that of the channel The distance apart of these pairs of poles is generally 50 feet for a slow current and 100 feet to 150 feet for a more rapid stream If more than one velocity observation is required in the cross section ropes should be stretched between the two pairs of poles clear of the water on which the centres of the divisions in which the water section is divided should be clearly indicated by attaching a red streamer or other wise The floats discs of wood for surface velocities or the weighted rods as already described for mean velocity observation are let go by an attendant at the proper place some short distance above the first pair of poles When the float crosses the first line of sight the time is noted and again when the float crosses the second line of sight Stop watches are now invariably employed for this purpose reading to half seconds At least five observations should be taken the mean being adopted as the true velocity If a float strays from the line it is expected to follow that observation should be cancelled

(37) In the case just described the water section is first of all divided into divisions but particularly in taking large rivers the width of the divisions may be arranged in accordance with the actual course taken by the floats The method of taking the discharges of the Nile illustrates this system and the description obtained from Egyptian Irrigation is as follows —

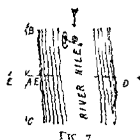


Fig 7

A site was chosen where the river was fairly straight for fully 2 kilometres the most uniform cross section of the river was found by taking a large number of rough cross-sections A peg was fixed in the bank opposite this section

In the accompanying plan A is thus fixed point

Through A a line B A C was marked exactly parallel to the direction of the river making AB and AC each equal to about the observed width of the river A theodolite or plane table was put up at A, and the point D across the river on the line AD at right angles to BC was fixed Since AB

and AC were both capable of being measured, they were measured, and with the aid of the theodolite or plane table the length AD was obtained

"In the line AD, or its continuation, flags were put up at E and E_1 to direct the man in the steamer on the Nile The theodolite or plane table was now put up at B or D and observations made on a steamer or boat on the Nile which as it got on the line AD, threw out a signal and took a sounding either with a line or sounding-rod The boat or steamer traversed the whole section, and if any gaps were left without soundings, the plane table showed where they were, and they could be immediately filled in While all this was going on the nearest river gauge was being observed, and also a temporary gauge erected at the site of the discharge site The two were afterwards connected by levelling This completed the observations necessary for the cross section, which was now plotted (*vide* Fig 8)

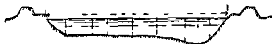


Fig 8

(38) "The surface velocity observations were now made

Two lines, HJ and KL (Fig 9) were fixed the former 50 metres up stream of AD and the latter 50 metres down stream Between these lines the surface floats were observed A theodolite or plane table was fixed at H and the boat or steamer was sent up stream with the floats The observer stood at H, and another at A The boat dropped a float into the stream at a distance of about 50 metres up stream of the line HJ at a convenient point and the theodolite or plane table at A followed it until the man at H called out, on its crossing the line HJ It was then observed and recorded This was repeated as the float crossed the line KL



Fig 9

"The observers at H and K noted the time When a sufficient number of well placed velocities over the half of the river near A were observed, observations for the other half were made from D The field work was now over On the cross section Fig 10, the points, E F G and C, where the velocities were observed, were plotted

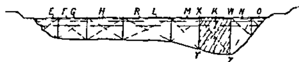


Fig 10

"The cross-section was divided into a number of suitable sections, each ruled by one or more observed velocities Each section was calculated separately, for instance, the section $XYZW$ Its area was $XYZW$, its wetted perimeter was YZ, its hydraulic mean depth was $\frac{XYZW}{YZ}$ and the rest followed from Bazin s coefficients and tables

(39) 'In addition to the discharges obtained from surface velocity observations, others were calculated from surface slope observations A very careful cross section of the river was taken as before, on a straight reach of some three or four kilometres, and at a point where the cross section was fairly uniform, showing that the river here was flowing normally A section taken in winter was found more accurate than in flood There was enough water in the river to preserve its normal slope of water surface, and not enough to render the taking of soundings difficult The water surface was compared with the nearest fixed gauge of the river The section was very carefully plotted and the water surface drawn on it, with the corresponding gauge readings written against it Horizontal lines (*vide* Fig 11), 1 metre or half a metre apart, were then plotted on this section, corresponding to the different gauge readings The cross sections and hydraulic mean depths (*i e*, A and R) were now calculated for each gauge reading For the slope of the river surface a long reach of 50 kilometres was taken, as the chances of error were very much less than they would have been if 3 or 4 kilometres had been taken This was easily effected on the Nile, as there were carefully levelled gauges 50 kilometres apart



Fig 11

The mean slope on this reach of 50 kilometres was taken as the slope for calculating the discharges Of course on curves the slope varies, but since the cross section had been taken in a carefully chosen normal site, the mean slope would refer to it From the calculated velocities and discharges, a discharge and velocity diagram was made This diagram was checked frequently by surface velocity observed as already described'

From the above the importance of taking the longitudinal section for some miles in length to ensure accuracy in the average surface slope which is naturally variable is demonstrated The same would apply with equal force to a longitudinal section of the bed of a river taken when dry if the water slope cannot be otherwise ascertained

Afflux

(40) The calculation for afflux, *i e*, the rise of the water surface in a river due to an obstruction in such a weir, is best arrived at by use of Tables II and IV when once the value of the velocity of approach or $Q \div A_1$ has been decided on

In a free overfall the afflux is the depth of film passing, plus that of the tail water surface below crest of weir In a submerged fall, H is the afflux To obtain d the flood discharge, which is a known quantity, should first be divided by the length of the weir, and the quotient will be the discharge per foot run The next procedure is to find by interpolation in Table II that value of d which corresponds to the discharge and the velocity of approach

(41) For example,* supposing a weir to be built across a channel. Let its length be 30 feet and let the flood discharge of the river be estimated at 600 cubic feet per second. Suppose the area of the flood waterway is 228 square feet, whence the mean velocity $V = \frac{Q}{A}$ and Va, the velocity of approach $= \frac{Q}{A_1}$. Assuming $A = A_1$, $Va = \frac{600}{228} = 2.63$ feet. The unit weir discharge being 20 feet, by Table II, under $V = 2\frac{1}{2}$ we see that d will lie between $d = 3$ and $d = 3\frac{1}{2}$, where the discharges in that column are 18.064 and 22.631 respectively. Their difference is 4.567, and that between the lower and the given discharge is 1.936 feet. Therefore the addendum to $d = 3$ will be $\frac{1{,}936}{4{,}567} \times \frac{1}{2} = .21$ nearly, and d will $= 3.21$. Allowing roughly for the difference between 2.63 the actual velocity, and 2.5 which has been used, d will probably equal 3 feet. The afflux will be 3 feet plus height of crest above tail water.

(42) The afflux in the case of a submerged * weir is found by the same process. Let us assume the crest of the weir to be 3 feet below the flood level of the river below the weir. The discharge, as before, is 20 feet per foot run, d will equal $H + 3$. If H be taken as 1, $d = 4$ and the corresponding discharge in Table IV, Series II, for $V = 2\frac{1}{2}$ is 17.9 cubic feet. H must therefore exceed 1 foot. If taken as 1.5 the corresponding discharge of $d = 4\frac{1}{2}$ in Table IV, Series III, is 22.9 cubic feet, which is too large, hence the value of H must lie between $1\frac{1}{2}$ and 1, and as 20 is a mean between the two values 22.9 and 17.9 the value of H may roughly be taken as $1\frac{1}{4}$.† For practical requirements the afflux need only be calculated to the nearest half-foot.

(45) Two rather complicated problems, connected firstly with the rise of a river or stream up stream of an obstacle, and secondly with the fall of the surface due to an opening, producing a draft on the discharge, have been dealt with by M. Bresse, a French mathematician. For the resulting formulas and tables we are indebted to that excellent work "A Treatise on Hydraulics" by Professor Merriman (New York).

* For a "free" fall a simpler calculation is: discharge per foot run $= \frac{600}{30} = 20 = 3.33H^{3/2}$. Whence $H = 3.3$ feet. And afflux $= (H) +$ elevation of crest of weir above tail water.

This ignores head due to velocity of approach which however is negligible since velocity of approach is only $\frac{600}{228} = 2.63$ feet per second, so that $\frac{2.63^2}{64.4} = 0.1$ foot only.—ED.

† A simpler calculation is that given in Molesworth's Pocket Book, viz.:—

$$R = \left(\frac{V^2}{58.6} + 0.05\right)\left\{\left(\frac{A}{a}\right)^2 - 1\right\}$$

where V = velocity of stream previous to obstruction (feet per second)
A = sectional area of stream unobstructed (square feet)
a = " " at obstruction (square feet)

$$R = \text{rise of water caused by obstruction} = \left(\frac{V^2}{58.6} + 0.05\right)\left\{\left(\frac{A}{a}\right)^2 - 1\right\} =$$

$$\left(\frac{2.63^2}{58.6} + 0.05\right)\left\{\left(\frac{228}{30 + 3}\right)^2 - 1\right\} = 0.17\,(6.4 - 1) = 0.92 \text{ foot.—ED.}$$

In that work they are termed the "backwater surface curve" and the "drop down" curve The latter appears a rather crude designation, it will be termed the 'falling surface curve"

Fig 12 is a diagram explanatory of the backwater surface curve *

BB is the sloping bed of the river assumed uniform and with a uniform width so that the depth D can be taken to represent the area per foot run as well as the hydraulic mean depth *CC* is the ordinary surface parallel to the bed *BB* The obstruction of the weir wall causes a rise in the surface which can be found by the method explained in the last paragraphs The

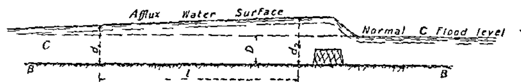


FIG 12 —Diagram of Backwater Surface Curve

depth of afflux level is designated d_2 and the depth up stream at a distance l above W is d_1 S is the slope Now l is found, when the other values are given by the following formula

$$l = \frac{d_2 - d_1}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right)\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right] \qquad (17)$$

Table VI gives the values of the decimal fraction $\frac{D}{d}$ and the corresponding value of the function of $\frac{d}{D}$

(46) The greater depth d_2 need not be at the weir itself, but anywhere above it and d_1 is the depth l feet above it

For example an actual case will be taken At Narora Weir (Figs 3 and 4 Chap VI) the afflux level is 18 feet above normal river bed and the depth below the weir is 16 feet † Thus $D = 16$ $d_2 = 18$ feet and d_1 will be taken as 17 feet The slope of the Ganges River is about 1 in 10 000 ‡ $\frac{1}{S}$ therefore is 10 000 Required to find at what distance above the weir the depth of the water will be 17 feet *i e* 1 foot rise above former flood levels

Here $\frac{D}{d_2} = \frac{16}{18} = 89$ and by reference to Table VI the corresponding value of $\phi\left(\frac{d_2}{D}\right)$ is 6173 in the same way $\frac{D}{d_1}$ is $\frac{16}{17} = 941$, and $\phi\left(\frac{d_1}{D}\right)$ is 82 nearly, c is taken as 70 and g the gravity sign is 32 and $\frac{(100c)^2}{g} = 153$ Then

* See Editor s Notes at the end of this Chapter —ED

† Fig 12 is not correctly drawn Compare Fig 3 Chap VI Height of weir is 10 feet only The depth of tail water is 16 feet Height of weir does not enter into the calculation —ED

‡ When in high flood the slope is about 1 in 4 000 —ED

$$l = 10{,}000\ (18 - 17) + 16\ (10{,}000 - 153)\ (820 - 617)$$
$$l = 10{,}000 + 157{,}552 \times 203$$
$$= 41{,}983 \text{ feet or nearly 8 miles}$$

(47) Another example will be given of the same formula in which d_1 is required and l is given In this case various values must be tried of $\frac{D}{d_1}$ and of $\phi\left(\frac{d_1}{D}\right)$ until the right-hand expression of the equation equals the left

TABLE VI VALUES OF THE BACKWATER FUNCTION

$\frac{D}{d}$	$\phi\left(\frac{d}{D}\right)$	$\frac{D}{l}$	$\phi\left(\frac{l}{D}\right)$	$\frac{D}{l}$	$\phi\left(\frac{l}{D}\right)$	$\frac{D}{l}$	$\phi\left(\frac{l}{D}\right)$
1	∞	0 954	0 9073	0 845	0 5037	0 61	0 2058
0 999	2 1834	952	8931	840	4932	60	1980
998	1 9523	950	8795	835	4831	59	1905
997	1 8172	948	8665	830	4733	58	1832
996	1 7213	946	8539	825	4637	57	1761
995	1 6469	944	8418	820	4544	56	1692
994	1 5861	942	8301	815	4454	55	1625
993	1 5348	940	8188	810	4367	54	1560
992	1 4902	938	8079	805	4281	53	1497
991	1 4510	936	7973	800	4198	52	1435
990	1 4159	934	7871	795	4117	51	13[illegible]6
989	1 3841	932	7772	790	4039	50	1318
988	1 3551	930	7675	785	3962	49	1262
987	1 3284	928	7581	780	3886	48	1207
986	1 3037	926	7490	775	3813	47	1154
985	1 2807	924	7401	770	3741	46	1102
984	1 2592	922	7315	765	3671	45	105[illegible]
983	1 2390	920	7231	760	3603	44	1003
982	1 2199	918	7149	755	3536	43	0995
981	1 2019	916	7069	750	3470	42	0909
980	1 1848	914	6990	745	3406	41	0865
979	1 1686	912	6914	740	3343	40	0821
978	1 1531	910	6839	735	3282	39	0779
977	1 1383	908	6766	730	3221	38	0738
976	1 1241	906	6695	725	3162	37	0699
975	1 1105	904	6625	720	3104	36	0660
974	1 0974	902	6556	715	3047	35	0623
973	1 0848	900	6489	710	2991	34	0587
972	1 0727	895	6327	705	2937	33	0[illegible]3
971	1 0610	890	6173	70	2883	32	0519
970	1 0497	885	6025	69	2778	30	0455
·968	1 0282	880	5884	68	2677	28	0395
966	1 0080	875	5749	67	2580	25	0314
964	0 9890	870	5619	66	2486	20	0201
962	9709	865	5494	65	2395	15	0113
960	9539	860	5374	64	2306	10	0050
958	9376	855	5258	63	2221	05	0015
·956	·9221	850	5146	62	2138	00	0000

This case will be taken in connection with Okhla Weir (Fig 16 Chap VI)

The given values are as follows —

$$D = 14\ 7 \quad d_2 = 17\ 7 \quad \frac{D}{d_2} = 83 \text{ and } \phi\left(\frac{d_2}{D}\right) = 4733 \text{ (from Table VI)}$$

l will be taken as 3 miles or 15 840 feet $100c$ as 170 and $\frac{100c^2}{g} = 153$ $S = 001$ The equation will then be 15 840 = 177 000 − 10 000 d_1 + 14 7 (9 847) $\left[\phi\left(\frac{d_1}{D}\right) - 47338\right]$

This worked out

$$10\ 000 d_1 - 144\ 750\left[\phi\left(\frac{d_1}{D}\right)\right] = 177\ 000 - 84\ 348 = 92\ 652$$

Let a trial value of d_1 be 17 feet then the equation becomes

$$170\ 000 - 144\ 750 \times 5494 = 90\ 415$$

which is as near as it possibly can be to the proper value which is 92 650

The value 5494 is $\phi \frac{d_1}{D}$ when $\frac{D}{d_1} = \frac{14\ 7}{17} = 865$ and is obtained from Table VI opposite 865

Thus 3 miles above the weir the flood afflux level will be 17 − 14 7 = 2 3 above the original river flood level

The Falling Surface Curve

(48) When a sudden fall occurs in a stream such as a drop in the bed as in canal fall which has not a raised or narrowed crest or from a side opening

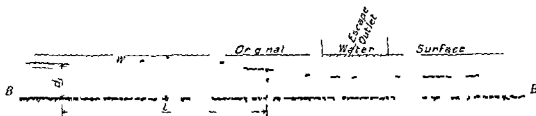


Fig 13 —Diagram of Falling Surface Curve

as a canal escape which draws the whole discharge away from the channel the water surface for a long distance above it is concave to the bed

This is explained in Fig 13

In this figure d_1 the greater value and d_2 the less are supposed to be depths of the water after the stream has settled down to its new regimen and has what is termed a steady though non uniform flow and let l be the distance between them Then the principle of formula (17) will apply in this case as well the only difference being due to d_1 having the higher value It stands as below

$$l = -\frac{d_1 - d_2}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right)\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right] \qquad (18)$$

(49) An example of the working of this formula will be taken from Merriman's "Hydraulics"

Take a canal 10 feet deep having a coefficient $100c$ equal to 80 and let the slope of the bed be $1 - 5{,}000$ and its surface slope be the same as when

TABLE VII —VALUES OF THE FALLING SURFACE FUNCTION

$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$	$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$	$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$	$\frac{d}{D}$	$\phi\left(\frac{d}{D}\right)$
1·	∞	0 954	0 8916	0 845	0 4478	0 61	0 0454
0 999	2 1831	952	8767	840	4353	60	0325
998	1 9517	950	8624	835	4232	59	0199
997	1 8162	948	8487	830	4114	58	+ 0 0074
996	1 7206	946	8354	825	3988	57	— 0 0050
995	1 6452	944	9226	820	3886	56	— 0172
994	1 5841	942	8102	815	3776	55	— 0293
993	1 5324	940	7982	810	3668	54	— 0412
992	1 4876	938	7866	805	3562	53	— 0530
991	1 4486	936	7753	800	3459	52	— 0647
990	1 4125	934	7643	795	3357	51	— 0763
989	1 3804	932	7537	790	3258	50	— 0878
988	1 3511	930	7433	785	3160	49	— 0991
987	1 3241	928	7332	780	3064	48	— 1104
986	1 2990	926	7234	775	2970	47	— 1216
985	1 2757	924	7138	770	2877	46	— 1327
984	1 2536	922	7045	765	2785	45	— 1438
983	1 23[illegible]3	920	6953	760	2696	44	— 1547
982	1 2139	918	6864	755	2607	43	— 1656
981	1 1955	916	6776	750	2520	42	— 1765
980	1 1781	914	6691	745	2434	41	— 1872
979	1 1615	912	6607	740	2350	40	— 1980
978	1 1457	910	6525	735	2260	39	— 2086
977	1 1305	908	6445	730	2184	38	— 2192
976	1 1160	906	6366	725	2102	37	— 2298
975	1 1020	904	6289	720	2022	36	— 2403
974	1 0886	902	6213	715	1943	35	— 2508
973	1 0757	900	6138	710	1864	34	— 2612
972	1 0632	895	5958	705	1787	33	— 2716
971	1 0512	890	5785	70	1711	32	— 2819
970	1 0396	885	5619	69	1560	30	— 3025
968	1 0174	880	5159	68	1413	28	— 3230
966	0 9965	875	5305	67	1268	25	— 3536
964	9767	870	5156	66	1127	20	— 4042
962	9580	865	5012	65	0987	15	— 4544
960	9402	860	4872	64	0851	10	— 5046
958	9233	855	4737	63	0716	05	— 5546
956	9071	850	4605	62	0584	00	— 6046

the water is in uniform flow Here $D = 10$ feet $(100c)^2 - g = 200$ and $1 - S = 5{,}000$ Then

$$l = -5{,}000\,(d_1 - d_2) + 48{,}000\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right]$$

It is required to find the distance between two points where $d_1 = 8$ feet and $d_2 = 7$ feet Here $\frac{d_1}{D} = 8$, for which $\phi\left(\frac{d_1}{D}\right) = 3459$ and $\frac{d_2}{D} = 7$, for which $\phi\left(\frac{d_2}{D}\right) = 1711$ Inserting these values in the equation, there is found $l = 3\ 390$ feet

Supply from run off of Rainfall

(52) In the cases of storage reservoirs it frequently happens that the supply is obtained, not by one large stream which can be gauged and its minimum and maximum supply estimated, but by numerous rills and streams which cannot well be separately dealt with Under such conditions the supply has to be estimated from the run off of the rainfall i e the least and also the greatest supply available from the catchm
subject has been well treated in the Mod = 90 415
Southern India has a very extensive sy
well as from direct canal irrigation no proper value which is 92,650
The following consists in great part of ext = 865, and is obtained from
work —

(53) *Rainfall* —Rainfall is the so flux level will be 17 — 14 7 = 2 3
purposes, and therefore a knowledge o
periods and of the effects produced b
all whose duty it is to design carry out *Curve*
works

The resulting discharge from rainfall has such as a drop in the bed, as
in a ca water to be utilised , (2) the water to crest or from a side opening
every irrigation work thes
nt
Origi ters are the fo
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tio

FIG 13 —Diagram of Falling Surface Curve

as a canal escape which draws the whole discharge away from the channel, the water surface for a long distance above it is concave to the bed

This is explained in Fig 13

In this figure d_1 the greater value, and d_2 the less, are supposed to be depths of the water after the stream has settled down to its new regimen and has what is termed a steady, though non uniform, flow, and let l be the distance between them Then the principle of formula (17) will apply in this case as well the only difference being due to d_1 having the higher value It stands as below

$$l = -\frac{d_1 - d_2}{S} + D\left(\frac{1}{S} - \frac{(100c)^2}{g}\right)\left[\phi\left(\frac{d_1}{D}\right) - \phi\left(\frac{d_2}{D}\right)\right] \quad . \quad ($$

hich case the proportion of the whole discharge intercepted may be large or small, or it may be utilised by means of irrigation canals or channels, with or without the aid of an anicut or weir to raise the water level in the watercourse, which may be the immediate source of supply The water so drawn off may be used for what is termed direct irrigation, as in the supply of canals or for that of tanks In all cases, however the whole quantity of water discharged by the basin has to be considered and divided into two parts the one to be used for irrigation and the other to be safely passed on or discharged to waste

(55) *Maximum Discharge of Catchment Basin* —Upon a right estimate of the greatest quantity of water liable to be discharged by a catchment depend the safety of the works which may exist, or which it may ...uct, to utilise a part of the water for irrigation purposes ...tion of this quantity is not possible, owing ..., under which heavy rain storms occur, ...icable, if a right use be made of available ... judgment of the quantity of water which ...d then, by allowing a reasonable margin for ...uisite safety may be secured

...harge Formulas —The first data to be ... the records of the levels and discharges ... the past These data require to be very ...d by comparison of the evidence obtained ... it has been decided what in the case of a ...d levels were it is necessary to ascertain from ...er that discharge is likely to have approximated ...xpected ...n conclusively that the flood discharge from large ...ly much less than that from small basins This fact ...duction of formulas to assist in the determination of ...d if judiciously used they are of considerable value for ... as a check on the results obtained from local observation

...hould be borne in mind however that though such formulas ...fidently used for the practical settlement of questions connected ...design of works when the conditions of their applicability have ...termined on adequate data they are otherwise to be relied on ...s a guide to the reasonableness or the reverse of the conclusions ...ndently arrived at The formulas which have been in use are the ...wing —

997			
996	1 7200		
995	1 6457	944	
994	1 5541	942	8.
993	1 5374	940	77
992	1 4576	938	7
991	1 4456	936	
990	1 4125	934	
989	1 3804	932	
988	1 3511	930	7455
987	1 3241	928	7332
986	1 2990	926	7-3
985	1 [illegible] 57	924	-1
984	1 2535	922	7(
983	1 2373	920	
982	1 2139	918	
981	1 1[illegible]55	916	
980	1 1[illegible]1	914	
979	1 1615	912	
978	1 1457	91[illegible]	
977	1 1305	9[illegible]	
976	1 11[illegible]0		
975	1 1020		
974	1 0886		
973	1 0[illegible]57		
972	1 0532		
971	1 0512		
970	1 039[illegible]		
968	1 01		
966	0 9[illegible]		
964			
962			
960			
958			
956			

(1) Ryves formula $Q = k_1\ 100\ K^{\frac{2}{3}}$ (20)

(2) Dickens " $Q = k\ 100\ K^{\frac{3}{4}}$ (21)

(3) Fanning's " $Q = \ 200\ K^{\frac{5}{6}}$ (22)

in which K represents the area of the catchment basin in square miles, k is a coefficient depending for its value upon rainfall, soil, slope of ground forming the basin, etc, and Q is the resulting discharge, which is usually taken in terms of cubic feet a second The first of these formulas assumes that the discharge from catchment basins of differing areas varies as the cube root of the square of the area, while in the second the variation is supposed to be as the fourth root of the cube of the area The general result is, in the former case a much more rapid diminution of proportionate discharge as the area increases than in the latter case The data for determining which is the more generally applicable formula for Southern India do not as yet exist, and either may be usefully employed under the restrictions above noticed as necessary. The following additional caution should, however, not be lost sight of No such formula can be strictly applicable with the same coefficient to areas of various sizes even in the same part of the country and within the influence of the same intensity of rainfall, unless the other circumstances, such as slope of the ground, description of the soil etc, be approximately similar

(58) The values of $K^{\frac{2}{3}}$, $K^{\frac{3}{4}}$ for areas from 1 square mile to 50,000 square miles are given in Table VIII The chief difficulty will be found in the selection of a suitable coefficient For the comparatively limited areas in the coast districts where the country is flat, and the drainage takes a longer time to run off $k = 4$ or 5 have been found to be suitable coefficients in Ryves formula and 6 75 is a suitable coefficient for limited areas near the hills

(59) The two Tables herewith given are (1) the discharges in thousands of cubic feet per second calculated from the two formulas, Ryves' $Q = k_1 100 K^{\frac{2}{3}}$ and Dickens' $Q = k 100 K^{\frac{3}{4}}$ In both the formulas for an area of 1 square mile the discharge is equal to the coefficient employed, *i e*, the run off is equal to the whole rainfall The first square mile is the base of the formulas, as the area increases, the proportion of run-off becomes less *in one formula as $K^{\frac{2}{3}}$ and the other as $K^{\frac{3}{4}}$ To utilise the Tables properly,* *i e*, to find the proper coefficient to be used, the method in Madras is to adopt as base an area of 5 square miles, in which area the maximum recorded rainfall is precipitated, this may be anywhere in a catchment basin It is rightly assumed that heavy cyclonic storms only occur over a very limited area In Table IX the run-off from the areas in square miles is converted from Table VIII into inches deep of run-off But 5 square miles being taken as the base, this run-off will be equal to the rainfall The examples on p 137 are taken from the "Madras Manual"

(60) 'For example, suppose the greatest recorded rainfall within or near a catchment basin under investigation to have been 11 inches The nearest run-off and rainfall to this in the line of 5 square miles (Table IX.) is 10 86 inches under coefficient 500 for Ryves' formula, and about midway between 400 and 500 for Dickens' formula Were no other data available,

Areas in Square Miles K	$Q = k_1 100 K^{\frac{3}{4}}$							$Q = k 100 K^{\frac{3}{4}}$						
	Coefficients (Ryves) (k_1)							Coefficients (Dickens) (k)						
1	0.200	0.300	0.400	0.500	0.600	0.700	0.800	0.200	0.300	0.400	0.500	0.600	0.700	0.800
5	0.584	0.876	1.168	1.400	1.752	2.044	2.336	0.668	1.002	1.336	1.670	2.004	2.338	2.672
10	0.928	1.392	1.856	2.320	2.784	3.248	3.712	1.121	1.686	2.248	2.810	3.372	3.934	4.496
25	1.710	2.565	3.420	4.275	5.130	5.985	6.840	2.236	3.354	4.472	5.590	6.708	7.826	8.944
50	2.714	4.071	5.428	6.785	8.142	9.499	10.86	3.760	5.640	7.520	9.400	11.28	13.16	15.04
100	4.308	6.462	8.616	10.77	12.92	15.08	17.23	6.324	9.486	12.65	15.81	18.97	22.14	25.30
250	7.938	11.91	15.88	19.84	23.81	27.79	31.76	12.57	18.86	25.15	31.43	37.72	44.01	50.30
500	12.60	18.90	25.20	31.50	37.80	44.10	50.40	21.15	31.72	42.29	52.86	63.44	74.01	84.58
1 000	20.00	30.00	40.00	50.00	60.00	70.00	80.00	35.57	53.35	71.13	88.91	106.7	124.5	142.3
2 500	36.84	55.26	73.68	92.10	110.5	128.9	147.4	70.71	106.1	141.4	176.8	212.2	247.5	282.8
5 000	58.48	87.72	117.0	146.2	175.4	204.7	234.0	118.9	178.4	237.8	297.3	356.8	416.2	475.6
10 000	92.83	139.2	185.7	232.1	275.4	324.9	371.4	200.0	300.0	400.0	500.0	600.0	700.0	800.0
25 000	171.0	256.5	342.0	427.5	513.0	598.5	684.0	337.6	596.5	705.3	994.1	1 193	1 392	1 591
50 000	271.4	407.1	542.9	678.5	814.2	950.0	1 086	668.7	1 003	1 337	1 672	2 006	2 340	2 674

TABLE IX.—(d) i.e., RUN OFF IN INCHES DEEP FROM AREAS IN SQUARE MILES IN 24 HOURS.

$$d = \frac{9Q}{242K} \quad K = \text{areas in square miles.}$$

Areas in Square Miles K	$Q = k_1 100 K^{\frac{3}{4}}$							$Q = k 100 K^{\frac{3}{4}}$						
	Coefficients (Ryves) (k_1)							Coefficients (Dickens) (k)						
1	7.43	11.45	14.87	18.59	22.30	26.02	29.74	7.43	11.15	14.87	18.59	22.30	26.02	29.74
5	4.34	6.52	8.69	10.86	13.03	15.20	17.38	4.96	7.44	9.92	12.40	14.88	17.36	19.84
10	3.35	5.03	6.71	8.39	10.06	11.74	13.42	4.18	6.27	8.36	10.45	12.54	14.63	16.72
25	2.54	3.82	5.09	6.36	7.63	8.90	10.18	3.33	4.49	6.66	8.33	9.98	11.65	13.33
50	2.02	3.03	4.04	5.05	6.06	7.07	8.08	2.79	4.19	5.59	6.99	8.38	9.78	11.18
100	1.60	2.40	3.20	4.00	4.80	5.60	6.40	2.35	3.52	4.60	5.78	7.04	8.12	9.20
250	1.18	1.77	2.36	2.95	3.54	4.13	4.72	1.67	2.81	3.75	4.69	5.62	6.55	7.50
500	0.93	1.40	1.87	2.34	2.80	3.27	3.74	1.57	2.35	3.14	3.93	4.70	5.49	6.28
1 000	0.74	1.11	1.48	1.85	2.22	2.51	2.96	1.32	1.98	2.64	3.30	3.96	4.62	5.28
2 500	0.54	0.81	1.09	1.36	1.63	1.90	2.18	1.05	1.57	2.10	2.63	3.14	3.67	4.20
5 000	0.43	0.65	0.87	1.09	1.30	1.52	1.74	0.88	1.32	1.76	2.21	2.64	3.08	3.52
10 000	0.34	0.51	0.69	0.86	1.03	1.20	1.38	0.74	1.11	1.48	1.86	2.22	2.59	2.96
25 000	0.25	0.38	0.51	0.64	0.76	0.89	1.02	0.59	0.88	1.18	1.47	1.76	2.06	2.36
50 000	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.49	0.74	0.99	1.24	1.48	1.73	1.98

in which K represents the area of the catchment basin in square miles, k is a coefficient depending for its value upon rainfall, soil, slope of ground forming the basin, etc, and Q is the resulting discharge, which is usually taken in terms of cubic feet a second The first of these formulas assumes that the discharge from catchment basins of differing areas varies as the cube root of the square of the area, while in the second the variation is supposed to be as the fourth root of the cube of the area The general result is, in the former case a much more rapid diminution of proportionate discharge as the area increases than in the latter case The data fo determining which is the more generally applicable formula for Souther India do not as yet exist, and either may be usefully employed under th restrictions above noticed as necessary. The
should, however, not be lost sight
applicable with the same coefficien
same part of the country and withi
rainfall, unless the other c
description of the soil etc

TABLE XII—TABLE OF TOTAL MONSOON RAINFALL AND ESTIMATED RUN-OFF AND YIELD PER SQUARE MILE FROM CATCHMENT AREAS IN NAGPUR (C P INDIA)

1	2	3	4	5	6	7	8	9	10
	Good Catchment			Average Catchment			Bad Catchment		
Total Monsoon Rainfall in inches.	Percentage of Run-off to Rainfall	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in m. c. f.	Per cent. of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in m. c. f.	Per cent. of Run-off to Rainfall	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in m. c. f.
1	0·1	0·001	0·00	0·1	0·001	0·00	0·05	0·0005	0·001
2	0	0·004	0·00	0 5	0·0 3	0·006	0·1	0·00	0·004
3	0·4	0·0	0·0	0·3	0·009	0·0	0·2	0·006	0·014
4	0	0·0 5	0·065	0·5	0·0 1	0·048	0·3	0·014	0·03
5	1·0	0·050	0· 6	0·	0·03	0·0	0·5	0·0 5	0·058
6	1·5	0·090	0· 9	1·1	0·06	0·1 6	0·7	0·045	0·104
7	1	0·14	0·341	1·5	0· 10	0· 55	1·0	0·0 3	0·
8	2·8	0· 4	0·	2·1	0· 6	0· 0	1·4	0·1 2	0· 6
9	3·5	0·3 5	0· 32	6	0· 5	0· 4	1·7	0·15	0·366
1	4·3	0·430	0· 3	7	0	0· 4	2·1	0· 5	0·4
11	5	0·	1·3	3·9	0·4	0·006	6	0· 6	0·664
1	6	0· 44	1·	4·6	0	1· 06	3·1	0·3	0· 4
13	2	0· 5	4	4	0·	1·6	3·6	0·4 8	1·
14	8·3	1·1	6	6·2	0	4	4·1	0· 8	1· 4
15	9·4	1·40	3· 6	0	1· 5	45	4·7	0· 5	1·0 8
15	10	1·600	3·	8	1·	9	5·2	0· 4	1·95
1	1 6	1·92	4·5	8	4 9	3·435	5·8	0·9 6	0
1	12·5	24	5·353	9·6	1· 5	4· 4	6·4	1	6 6
9	13·9	·641	6· 35	10·4	1·9	4·6 1	6·9	1·3	3·0
0	0	3·000	6· 0	5	5	5·2	5	1· 0	3·48
21	1 1	3· 81	5	0	2·5 5	5·591	0	1·0	3·9
	1 3	3· 06	8· 4	1·	2· 4	6·6 1	8·6	1·003	4·4
3	15·4	4	9· 2	5	3· 4	4	9	6	4·96
4	9	4·600	10·	14·5	0	4	9·	40	5·46
	2	5	4	4	6	3	0·3	5	5
5	8	3	13· 8	6	4·25	9 6	0·9	4	6· 4
2	9	6· 3	4· 64	1	4·6	1	1·4	3·	
28	21·0	6· 0	15·6 2	0	5·04	1 09	12·0	60	6
29		9	6·91	18·8	5·459	12·6	1 5	3·6 9	8·4
30	3	0	8	19·7	5·9	13· 4	13·1	3· 45	9· 6
3	4	94	1	5	6·3 0	14· 99	13·7	4· 4	9·60
3	28·5	9· 0	1	1·3	6·840	15·891	14·2	4·560	1 94
33	6	9· 8	613	2	7·3 6	7 0 9	14·8	4 4	1 46
34	8	10·4	24·3 9	23·1	7 4	18·246	15·4	5 6	1 04
3	3 9	1 65	939	23·9	8·3 3	19·454	5·9	5	1 06
3	0	1 8 0	600	4	8·0	00	6·5	5·04	
37	4	6	9 2	5	9·46	4	0	6 0	14· 6
38		4	6	4	0		1 6	6	5·5
39		4		3	10·647	5	2	8	
4			5		1	6 6		5	1 4 4
4		6		0			19	0	18
4		6				0	9·9	8	10·4
4	4 0		4	6			0·4	8	
44	4	0		5		99	0	0 4	
4	4	0			14 45	03	5	0	9
46	4								6
4	4 4	8	4	0		9			4 6
48	4 5	3		8	4				0
49	4 6	3 4	4 6		4	4 631	3·8	6 2	0
50	48·8	4 0	6	6·6	18	4 4	4·4	0	
51	49·9	449	3	4	9·0 6	44 1	4·9	4	6
52	0	6 0	6·6 1		9 8	46 08	5·5	0	
53	52·1	6 3	6 1	3 0	0	45 3	0	13 6	
54	53·3	8	66	39·9	5 6	49	6·6	4	4
55	54·4	9·9 0	6 0	40·8	44	5 13	2 2	4 00	3
56	55·5	3 0	5	4 6	3 0	4 53	7	1 4	6
57	66	6	4	4 4	6	3	3	6	5
58	7 8	33·5 4	3	4 3	43	4 2	28·9	6	5 4
59	8 9	4 5	8 34	44·1	6 03	6 5	9·4		4 6
60	60·0	36·000	83·6 5	45·0	2 0	6 6	30·0	00	4 5

accordingly Colonel Mullins in the Madras Irrigation Manual is of opinion that this proportion is excessive and that it should more correctly vary somewhat with the slope of bed of stream as well as being proportionate to the area of the catchment basin

(63) The shape of the catchment affects the coefficient, and attempts have been made to modify the coefficient so as to take the latter into account. There is a great deal of literature on this difficult subject, but nothing of a definite character The formula is $Q = k100 \frac{B}{L} K^{\frac{3}{4}}$ where B and L are breadth and length of the catchment (*vide* Jackson's "Hyd Manual," Chap I)

The minimum discharge from a catchment area may be considered as 10 per cent of the average rainfall

TABLE XIII —TABLE OF THE DISCHARGES OF A WASTE-WAY CHANNEL HAVING A BED-WIDTH OF 200 FEET AND A BED SLOPE OF 1 IN 100

1	2	3	4	5	6	7	8	9
Total Depth	Afflux Height d_1	Tail Depth d_2	Afflux Coefficient c_1	Channel Coefficient c_2	$\frac{D}{c_1 w \sqrt{2g}}$	$\sqrt{d_1}(d_2 + \frac{2}{3} d_1)$	Mean Velocity of Tail Channel.	Discharge, D
Feet	Feet	Feet					Feet per second	Cubic feet per second
1	0.30	0.70	0.60	41.00	0.495	0.495	3.40	476
2	0.74	1.26	0.60	52.10	1.510	1.505	5.78	1,457
3	1.24	1.76	0.60	59.00	2.85	2.87	7.78	2,741
4	1.70	2.30	0.62	64.4	4.47	4.46	9.66	4,444
5	2.15	2.85	0.64	68.8	6.34	6.29	11.42	6,510
6	2.63	3.37	0.66	72.0	8.29	8.29	13.03	8,784
7	3.08	3.92	0.68	75.0	10.46	10.45	14.55	11,407
8	3.52	4.48	0.70	77.5	12.80	12.84	16.04	14,374
9	3.95	5.05	0.72	79.5	15.23	15.28	17.41	17,585
10	4.36	5.64	0.74	81.4	17.87	17.87	18.80	21,210
11	4.77	6.23	0.76	83.1	20.56	20.51	20.11	25,057
12	5.17	6.83	0.78	84.5	23.34	23.34	21.38	29,203
13	5.54	7.46	0.80	85.8	26.24	26.20	22.57	33,668
14	6.00	8.00	0.80	86.8	29.44	29.40	23.61	37,775
15	6.47	8.53	0.80	87.7	32.65	32.61	24.56	41,893
16	6.94	9.06	0.80	88.6	36.03	36.00	25.51	46,236
17	7.42	9.58	0.80	89.3	39.47	39.52	26.43	50,645
18	7.86	10.14	0.80	90.0	43.10	43.06	27.28	55,304
19	8.33	10.67	0.80	90.7	46.88	46.81	28.19	60,157
20	8.80	11.20	0.80	91.3	50.67	50.63	29.03	65,035

When the capacity of a reservoir is well below its supply, it may fill [illegible]eral times during the rainy season, and its useful capacity will be pro[illegible]tionately greater than its estimated storage capacity

(64) Table XI is given in Strange's "Storage Reservoirs," and is [illegible]st useful for calculations of flood discharge based on actual values of [illegible]off in inches per hour

Table XII is based on observations taken near Nagpur, in the [illegible]vinces of India, a usually dry tract, by Sir Alexander Binnie, the Min Pro Inst C E

This gives the run-off and percentage for three catchments, classed as ᛁ, average, and bad The good catchment approximates to that observed ear Nagpur. It is based on general ideas, and will be of assistance when ı actual recorded stream or river discharges are available

(66) Table XIII, derived from Strange's "Storage Reservoirs," will ᛫ ıt calculations for depth of water flowing down bye-washes which are sc common a feature in many large storage and diversion dams

Discharge through Syphons

(67) The head or difference of levels above and below a syphon is generally required to be ascertained the mean velocity through the barrels being a given quantity This is sometimes taken at 15 feet per second in order to clear out all detritus, 8 feet being usual The head is found by the following ᵓormula —

$$H = M\frac{V^2}{2g} = 0155\ MV^2 = hM \qquad (23)$$

h being the head due to the velocity V, or $0155\ V^2$

In this expression $M = \left(1 + f_1 + f_2\frac{L}{R}\right)$ in which L = length of barrel, R = hydraulic mean depth, or $\frac{\text{Area}}{WP}$, f_1 is the coefficient for loss of head by entry, and varies from 08 for an ideal converging entrance to 505 for an abrupt change from a wide channel into a round pipe or square culvert This latter value, or some modification of it, is most suitable for masonry syphon barrels, f_2 the second coefficient, is that of friction on the sides of the barrel and $= a\left(1 + \frac{b}{R}\right)$.

Suitable values of a and of b must be selected from the Table below

Nature of Barrel	a	b
Iron pipes	00497	084
Cement plaster	00294	10
Ashlar or brickwork	00373	23
Rubble masonry	00471	82

If the converse to formula (23) is required

$$V = 8\ 025\sqrt{\frac{H}{M}} \qquad (24)$$

If velocity of approach be considered

$$H = \left(M \times \frac{V_2}{2g}\right) - \frac{V_a^2}{2g} \text{ or } = 0155\left(MV^2 - V_a^2\right) \qquad (25)$$

$$\text{Conversely } V = \sqrt{\frac{2g\ H + V_a^2}{M}} \quad . \ . \ . \ . \qquad (26)$$

For example, let the syphon barrels be 6 feet long by 4 feet deep, then $R = \frac{A}{WP} = \frac{24}{14} = 1.7$ nearly. Let $L = 170$ feet, then $\frac{L}{R} = 10$. Further let V be assumed as 15, f_1 as .5, a as .004 and b .23 from Table, then $f_2 = .004\left(1 + \frac{.23}{1.7}\right) = 1.545$.

By formula (23) $H = .0155 \times 1.545 \times 225 = 5.38$ feet.

If velocity of approach * of 3 feet per second be assumed, the above value of H will be reduced by $.0155 \times 9$ or by .1395.

H will then $= 5.14$ feet.

Useful Memoranda

(68) 1 cubic foot of water $= 62.4$ lbs. $= \frac{1}{36}$ ton.

1 ton of water $= 36$ cubic feet.

Inches of rainfall $= 53.3$ acre feet per square mile.

1 inch run off per hour per square mile $= 645.3$ second feet $= 1$ cubic foot per acre per second.

1 cubic foot per second $= 2$ acre feet (short name "cusec").

Feet per second $=$ .68 miles per hour.

1 acre foot $= 43,560$ cubic feet.

Pressure of water per square foot $= \frac{H}{36}$ tons.

Velocity of Water

(69) V = Theoretical velocity in feet per second.

g = Force of gravity $= 32.2$

$2g = 64.4$ $\qquad \sqrt{2g} = 8.025$ $\qquad \frac{1}{2g} = 0.0155$

H = Head of water in feet.

$V = \sqrt{2gH} = 8.025\sqrt{H}$

$H = \frac{V^2}{2g} = 0.0155\,V^2$

Editor's Notes

(G) *Velocity of Approach.*—This question lends itself to misconception and exaggeration. It is easier [illegible] with much nicety on paper [illegible] accuracy in practice. In the [illegible] in Fig. 1 we see that he reckons the increase of discharge, due to velocity of approach, to amount to $\frac{379.95 - 378.8}{378.8}$ = about one third of 1 per cent, a negligible consideration in practice.

* In the case of a syphon the flow on changing its direction loses the velocity of approach, which is converted into visible static head.—Ed.

In fact, in the case depicted in Fig 1, where dynamic head of flow is converted mostly into static head at the dam, the propriety of making separate allowance for velocity of approach is questionable In canals ranging up to 10,000 cusecs capacity the mean velocity is usually less than 3 5 feet per second, and the "head" due to such velocities amounts to about 1½ per cent of the water-depth—a dimension well within the ordinary margin of error of measurement of flowing water, whilst, in great rivers, wherein $\left(\frac{V^2}{2g}\right)$ may present a more respectable value, the difficulty of determination is correspondingly greater Thus, in the case of a great flood which passed over the weir at Khanki, on the River Chenab, in the Punjab, in July, 1903, the maximum discharge might be computed to be 460,000, or 530,000, or 620,000 cusecs, according as the mean velocity of approach was supposed to be 0, or 6, or 10 feet per second One engineer even put the figure so high as 750,000 cusecs, though the data of the case are so uncertain that it is impossible to be sure that the maximum discharge was more than say 550,000 The weir is about 4,000 feet long, apart from its sluices, which have 240 feet of waterway The river up stream of it may be, when in high flood, much wider, flowing, not in a single well-defined channel, but in several separate streams—converging, diverging, or interlacing—with shoals or islands between, and although these shoals, submerged in high flood, form part of the river bed, they may be, in places, at higher levels than the crest of the weir itself The weir is divided into eight bays of 500 feet, by piers and groynes, and the oblique set of different currents towards these groynes may affect the depth of water over crest at different places It was impossible to navigate the river during this high flood, or to measure its cross-sectional waterway or mean velocity The weir-crest was submerged by tail-water to a depth of 10½ feet, and the sluices to 18 feet The afflux, or difference of water level up stream and down stream of the work, was only 2 feet at right flank, and 3 feet on left flank, whilst water levels in midstream were unknown The higher water level recorded on the left flank may very likely have been due to a set of the current towards that flank and away from the other The gauge-readings, in fact, may have been the static representation, *inter alia*, of the velocity of approach. These and similar practical considerations may well make us chary of approaching the problem of head due to velocity of approach with a velocity of too much confidence in our head-work

(H) *The Backwater Surface Curve*—Par (45)—This calculation has, perhaps, more of academic interest than of practical utility as it rests upon imaginary data that are rarely met with in practice

The calculation assumes that the channel under consideration is of great and uniform width, and uniform slope and water-depth It assumes, also, that the water is not silt-laden Such conditions do not occur naturally in rivers, nor even in earthen canals The first effect of an obstruction, such as a weir, across a silt-bearing stream, is to diminish velocity and cause the water to drop its silt-burden, so that the theoretical water-depths up stream are altered by the very creation of afflux At Narora, on the River Ganges, for instance, the river bed is shoaled up above crest-level of weir in some places up stream of it, and scoured down to foundation-level elsewhere The course of the river up stream is, moreover, very tortuous (Fig 33a, Chapter VI), flowing in several channels, with shoals and islands

between them In such circumstances the calculation for backwater curve is too fanciful for practical utility

A curious instance of misconception in this matter of backwater-curve has been noticed recently in connection with the design of the Barrage which is now being constructed across the River Indus below the Sukkur Gorge

As depicted in the plan (Fig 14) the River Indus just above the town of Sukkur in Sind forces its way through a rocky gorge in a low range of hills in two channels split up by the rock island of Bhakkar The combined width of these two channels is only 1 400 feet at high flood level Further down stream the gorge gradually widens and at the point *B* two miles below the gorge *A* the river is about 3 000 feet wide One mile still further down stream at *C* it is 5 500 feet wide and it is here that the Barrage or Dam is being built with a waterway only

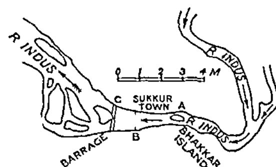


Fig 14

4 000 feet wide The Barrage engineers have calculated that this reduction of width from 5 000 feet to 4 000 will set up an afflux under maximum flood conditions of only 1 foot and reckoning by the ordinary formula for backwater curve they have concluded that the 1 foot afflux at *C* will diminish to only 3 or 4 inches at *B* and that it will vanish completely a little way up stream of *B* so that it will not affect the conditions of flow at the narrow gorge *A* The calculation overlooks the fact that the channel up stream of *C* is not of uniform width or depth or slope and that the water under high flood conditions will be heavily laden with sand Under such conditions the afflux instead of diminishing will increase upstreamwards in the narrowing channel Statistics recorded of water levels at *A* and *B* respectively show that under existing conditions every unit rise of water level at *B* is accompanied by a greater rise in the congested channel at *A* so that the further congestion set up at *C* by the Barrage of the future will undoubtedly take effect at *A* The importance of this consideration lies in the risk that exists of the Indus deserting the Bhakkar Gorge and changing its course to lower lying land which exists some miles to the right or left of it The critics of the Barrage Project opine that the tendency of the Barrage built down stream of the gorge *A* will be to increase the risk of river avulsion However that may be the case in point is a signal illustration of the difficulty of application of the backwater-curve calculation in practice

(k) *The Falling Surface Curve* Par (48) —What we have remarked above as to the academic nature of the backwater curve speculation applies also *mutatis mutandis* to the case of the Falling Surface Curve In an earthen channel such a condition could not exist for long It would set up violent erosion of the stream bed lowering the bed level so as to *relatively* elevate the masonry work at tail converting the latter into a weir increasing in height till equilibrium or normal current *regime* is attained This is what actually happened in the Ganges Canal in its earliest working days and the remedy was found in the construction of weirs or crest walls, on the brink of the masonry falls of the canal

CHAPTER VI

DIVERSION WEIRS ON SAND FOUNDATIONS

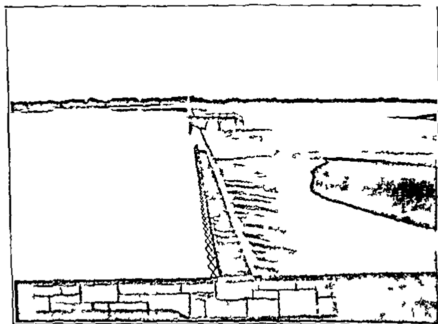


FIG 1a —Narora Weir Lower Ganges Canal

(1) A CLASS of weir peculiar to India includes those erected across the great rivers of the peninsula such as the Ganges the Jumna the Chenab the Jhelum and many others in Upper India and the Mahanuddee Son Kistna Godaveri and Penner in Bengal and Southern India These are naturally exclusively diversion weirs and are of no great height 10 or 12 feet above normal river bed level or low water level being generally the outside limit of their height What they lack in this respect is however made up not only in length but in width The weir over the Son River at Dehri is 2½ miles long and those spanning the great Godaveri River with its flood discharge of over 1 200 000 second feet are nearly as long The Okhla Weir is 250 feet wide and several others run this dimension very close Thus it is that these canal head works rank among the largest and most important in the whole world

There could hardly be a greater contrast between the narrow but immensely high American dams built over narrow rocky gorges amid wild and sterile surroundings and these long low Indian weirs which are generally situated among cultivated lands and a teeming population often amidst historic remains of great antiquity The main peculiarity of these

between them In such circumstances, the calculation for backwater-curve is too fanciful for practical utility

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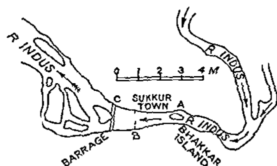


Fig 14

CHAPTER VI

DIVERSION WEIRS ON SAND FOUNDATIONS

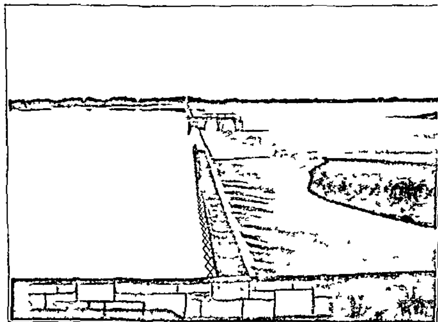


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Indian weirs is the nature of the foundations, which is mainly sand, down to great depths, perhaps 1 000 feet, or more

Up to a quite recent period the design of such works has been subject to great uncertainty, the sections being adjusted by failure and renewal to actual requirements

Now, however, that the principles regulating the stability of structures founded on a pervious and loose material such as sand are better understood, the practical application of these principles is simpler

A drowned diversion weir on sand, although its height is seldom over 10 feet above the normal river bed, is not only exposed to the destructive influences of a large river in flood, but its foundation, being necessarily the sandy bed of the river, is liable to be undermined and washed out by the hydrostatic pressure of the water upheld in rear of the weir In spite of these drawbacks, it is however, quite practicable to design a work of such outline as will successfully resist all these disintegrating forces, and remain a permanent and solid structure

(2) The principle which underlies the action of a head of water on a porous stratum of sand, over which a heavy impervious weight is imposed is analogous to that which obtains in a pipe under pressure

Fig 1 exemplifies the case with regard to a pipe-line *BC* leading out of a reservoir The head *H*, is the difference of levels between *A'*, a point

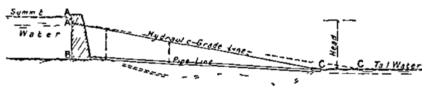


FIG 1 —Diagram of Pipe under Water Pressure

somewhat lower than A (the actual summit level) and, *C*, the level of the tail water beyond the outlet of the pipe The water having a free outlet at *C*, the line *A'C* is the hydraulic gradient or grade line The hydrostatic pressure in the pipe at any point is measured by vertical ordinates drawn from the centre of the pipe to the grade line *A'C* The uniform velocity of the water in the pipe is dependent directly on the head, and inversely on the frictional resistance of the sides of the pipe, that is on its length This supposes the pipe to be straight, or nearly so, and uniform in section

(3) We will now consider the case of an earthen embankment thrown across the sandy bed of a stream The pressure of the impounded water will naturally cause leakage beneath the impervious earthen base With a low depth of water impounded, it may well be understood that such leakage might be harmless, that is, the velocity of the percolating undercurrent would be insufficient to wash out the particles of sand composing the foundation of the dam When, however, the head is increased beyond a safe limit,

the so termed "piping" action will take place and continue until the dam is completely undermined

(4) The main determinating factor in the stability of the sand formation is evidently not the superimposed weight of the dam as the sand is incompressible, although a load in excess of the hydrostatic pressure must exercise a certain, though possibly undefined, salutary effect in opposing disintegration of the substratum However this may be, it is the enforced length of percolation or the so-called *creep* of the undercurrent that is the real determining influence

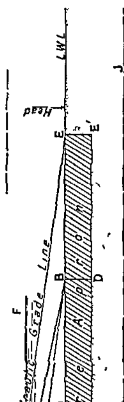


FIG 2 —Diagram of Weir with floor

(5) In the case of a capillary tube, the induced velocity is inversely proportional to its length In that under consideration the hydraulic condition being practically identical, it is the enforced percolation through the sand and the resulting friction against its particles as the water forces its way through this medium, that effects the reduction of the velocity of the undercurrent, and this frictional resistance is clearly directly proportional to the length of passage

In the example under consideration, this length of enforced percolation is evidently that of the impervious base of the dam The moment this obstruction is passed the water is free to rise out of the sand, and the hydrostatic pressure ceases

(6) It is evident, therefore, that to ensure safety from undermining, this length of enforced percolation, or creep, which will be symbolised by l, must be some multiple of the head H; and if a reliably safe value for this factor or coefficient can be found suitable to any particular class of sand, we shall be enabled to design any work on a sand foundation with perfect confidence with regard to stability If this coefficient be symbolised by c,

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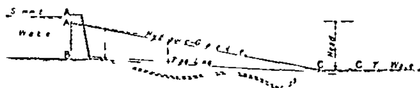


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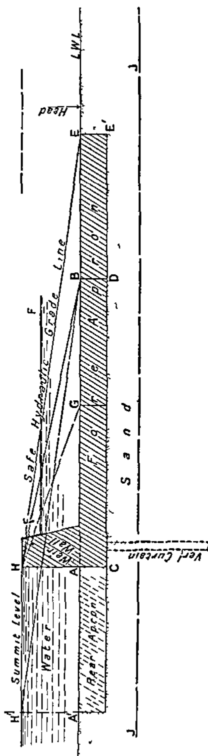


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then l, or the length of enforced percolation, will equal $c \times H$, H being the head of water. The coefficient c will vary in value with the quality of the sand.

(7) Fig. 2 represents a case similar in every respect to the last, only that instead of a dam of earth the obstruction consists of a vertical wall, termed the weir or drop-wall, having a horizontal impervious floor $ACDB$ attached thereto, which appendage is necessary to prevent erosion of the bed by the current of falling water when the weir is overtopped.

The level of the tail water is supposed to be that of the floor level, consequently the hydraulic gradient will be HB,* and as in the previous case of the pipe line the vertical ordinates of the triangle HAB will represent the upward hydrostatic pressure exercised on the base of the weir wall and of the floor.

(8) The safety of the structure is evidently dependent on the following points:—

First, the weir wall must be dimensioned so as to resist the overturning moment of the horizontal water pressure. This has been dealt with in a previous chapter.

Secondly, the thickness, i.e. the weight, of the apron or floor must be such that it will be safe from being blown up or fractured by the hydrostatic pressure. And thirdly, its base length or that of the enforced percolation l, must not be less than $c \times H$, or than the product of the coefficient c with the head H.

It is evident that the value of the coefficient c must vary with the nature of the sand substratum in accordance with its qualities of fineness or coarseness. Fine light sand will be closer in texture, passing less water under a given head than a coarser variety, but at the same time fine sand will be disintegrated and washed out under less pressure. Reliable values of c, on which the design mainly depends, can only be obtained experimentally, from artificial experiments, and classification of samples of sand taken from the sites of actual examples of weirs, and the records of failures due to insufficiency in length of percolation path. From these statistics a safe value of the relation of l to H, or of the coefficient c, which also defines the inclination of the hydraulic gradient, can be derived.

(9) The following values of c have been adopted for the classes of sand detailed below, the derivation of which will be explained later:—

Class I. River beds of light silt and mud, as the Nile. $c = 18$.

Class II. Fine micaceous sand, as in certain sites on Himalayan rivers and such as the Colorado in the United States. $c = 15$.

* To be accurate the hydraulic gradient $H'B$ should be drawn from some point H' lower than H on the vertical AH for the same reason that in Fig. 1 it is drawn from A' lower than A on the vertical AB.—ED.

Class III Coarse grained sands as in Central and South * India
$c = 12$

Class IV Boulders or shingle and gravel and sand mixed
c varies from 9 to 5 †

(10) In Fig 2, supposing that the width of the base of the apron and drop wall, or CD, is found just sufficient to produce equilibrium as regards length of percolation, then the hydraulic gradient will be HB The value of l thus provided will, however, be inadequate A factor of safety of say one half is required, so that the floor should be extended to E, AE being equal to $1\frac{1}{2}$ AB The apex of the triangle of pressure will then be moved to the point E, the hydraulic gradient being AE This extension provides increased safety from the danger from piping, but enlarges the area of hydrostatic upward pressure from the triangle HAB to HAE Consequently, in addition to the element of cost, there is a distinct disadvantage in lengthening the impervious floor beyond what is requisite to ensure an absolutely sufficient length of creep

Supposing the width of the floor to be reduced below the minimum CD or AB to AG, the hydraulic gradient will then be HG This reduces the area of the triangle of pressure to HAG, but failure may take place by piping, the sand substratum being gradually washed out, causing the floor to collapse

(11) The proper profile of the apron to resist the hydrostatic pressure is evidently that of a triangle, its base being at the root or toe of the drop wall In practice this will be a trapezoid, as the end thickness cannot well be reduced to zero Its weight can be conveniently represented by $t \times \rho$, t being the thickness and ρ the specific gravity of the material

The value of $t\rho$ should exceed that of the upward hydrostatic pressure by, say, one third This latter can be represented by the symbol $(H - h)$, H being the head acting on the base and h the neutralisation of pressure that is, of head, effected by the length of percolation up to this point Let l be this length, then $h = \frac{l}{c}$

(12) At this juncture a few points regarding loss of weight due to displacement of immersed parts of the weir require notice

With regard to the hydrostatic pressure on the base

In Fig 2, if a hole were supposed bored in the floor CE and a pipe inserted, the water would rise up as far as the hydraulic gradient HE The pressure acting on the base CE would therefore be greater than the triangle HAE—would, in fact, be the trapezoid $HCE'E$ This is accounted for by the addition to the external head, which is HA, of that due to the displacement of the floor

* We have no evidence of the coarseness of South Indian sands i e no evidence in the nature of physical analysis —Ed

† It is to be borne in mind that where Himalayan rivers issue from the mountains on to the alluvial plains their bed soil is of boulders shingle and coarse sand but that progressively downstreamwards the bed soil becomes gradually finer and lighter in proportion to distance from the mountains —Ed

To avoid confusion the extraneous or active head of water symbolised by H which always represents the difference of levels above and below a weir or regulator will be kept distinct from that due to displacement or immersion this latter pressure being allowed for by reduction in the effective weight of the immersed body

(13) A solid material immersed in water loses as is well known a proportion of its weight equal to the weight in water of the area immersed or if specific gravity be substituted for weight and if the letter ρ symbolise the specific gravity of a material when the same is immersed in water its specific gravity can be considered as if reduced by unity or to be $(\rho - 1)$

Thus if the specific gravity of a block of masonry be $2\frac{1}{4}$ when immersed the reduced weight can be expressed by $(2\frac{1}{4} - 1) = 1\frac{1}{4}$ *

(14) In Fig 2 when the low water level is at L the floor is immersed and its effective weight is as $t(\rho - 1)$ $t\rho$ t being the thickness When the water level rises to $I I$ the status is in no way altered as the excess of upward pressure above what it was at the lower level is compensated by an equivalent load of water lying above the floor

When the low water level is at CDF the weight is unimpaired when half way between F and F half of the floor will be considered as of lower specific gravity than the other half

If the water level were to fall below the level of the base of the floor as to JJ the sand substratum being porous when the latter is under pressure as from a head of water held up by the weir wall the water will rise up to the base of the impervious floor thus practically reducing the head from HJ to HC This is conditional on its being contained in the pressure area i e below the hydraulic gradient The head acting on the apron cannot be measured to any depth lower than the base of the latter

(15) Another most important point requiring notice is the relative position of the apron With regard to the drop wall in Fig 2 supposing the impervious floor were extended backwards to A and AA made $= BF$ then the action of the head of water is thrown back from H to H and the hydraulic gradient will be $H B$ parallel to $H L$ and the statical condition in no wise altered

Thus we see that the position of the drop wall with reference to the floor or apron is immaterial as regards foundation stability but on the other hand a certain length of solid apron in front of the drop wall is necessary to protect the bed from erosion so that only a proportion of the floor length can well be placed in rear of the drop wall

(16) This rear projection is termed the rear apron in contradistinction to the fore apron or floor proper lying in front of the drop wall The rear apron is in an advantageous position It is somewhat protected from

* It should be borne in mind however that ordinary brickwork and concrete when saturated with water may be nearly 20 per cent heavier than when dry —Ed

erosive* action except from cross currents, and for this reason can be constructed of less expensive material than the fore apron On the other hand, it must be impervious and must have a watertight connection † with the drop wall

(17) From the above, it would seem that the rear apron would be effective if only a thin, impervious layer, but weight is necessary to some extent in order to prevent the possibility of the undercurrent partaking of the nature of a surface current, and so losing the efficacy of the frictional resistance of the sand For the same reason it is always advisable, as has already been noted, to make the effective weight of the fore apron somewhat in excess of actual requirements as regards hydrostatic pressure

Weight in these submerged structures is always a desideratum, and is only limited by the necessities of cost

(18) We have seen that the rear apron has, as its chief function the neutralisation of statical pressure, whereas the fore apron or floor in addition to its functions in a hydrostatical sense has to provide a solid cushion for resistance to the erosive force of the current of falling water for which purpose also width is a necessity The width need not be all composed of impervious masonry, but can be formed in the lower portion, or talus, of loose riprap The total width is dependent on several considerations, and must remain more or less a matter of individual judgment, although certain approximate rules can be deduced from examination of sections of existing works of similar character which will prove reliable guides to the designer This point will come up again later, in pars (25) and (27)

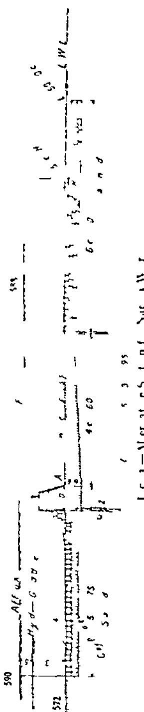
(19) The line of creep, we have hitherto considered has been a horizontal one If vertical depressions are placed below the base of the floor, or apron the line of percolation may be forced to follow round these obstructions and may not, as might be imagined take the line of least resistance Thus if an impervious line of sheet piling or a curtain wall of masonry as one of undersunk hollow blocks connected together by piling be inserted *below the floor, as near C in Fig 2 the line of creep may be measured down* one side of the vertical obstruction and up the other side The added length of creep will thus be *twice* the depth of the curtain

The insertion of a curtain wall of any kind is therefore a most valuable means of increasing the length of the enforced percolation and of thus immediately reducing the pressure at any part of the base The effect of vertical depressions in the floor on the hydraulic gradient line is to break the continuity of the hypothenuse of a triangle of pressure by steps each step being equal to the depth of the vertical obstruction divided by the coefficient or if the obstruction is sheet piling or a very narrow wall the depth at this point will be twice the above, or $2l - c$ It is however

* Yet at Narora it was completely scoured away See par (8) *infra*—Ed

† But at Okhla Weir on the River Jumna loose stone dumped into the stream up stream of the weir was sufficed for all purposes of efficiency —Ed

considered that the efficacy of curtain walls in this respect is dependent on their not being spaced nearer to each other than twice their depth.



(20) A practical example of the method of applying the principles already set forth will now be given in Fig. 3 under the conditions actually prevailing in the Narora Weir on the Ganges River. This work is of the direct overfall type which will be classified as type A. The data on which the design is based are as follows: Sand class 2—coefficient $c = 15$; H or difference of summit and of low water level—the latter always symbolised by the letters L W L = 13 feet; then the required lengths of creep or of l must be $c \times H = 15 \times 13 = 195$ *feet*. *We have now to decide what proportions of this length are to be placed in the fore apron, in the rear apron, and in the vertical sheet piling.* The minimum allowable width of the fore apron is naturally a leading consideration in this problem. This width of apron is affected in two ways: firstly, by the nature of the river bed represented by its coefficient c, and secondly, by the obstruction offered by the weir wall, *i.e.* by the height of the permanent crest excluding the crest shutters, which latter are only in use during low water. The height of the shutter crest above the floor, *i.e.* the depth of the drop, will be symbolised by H^s in order to differentiate it from H, the head of water. In some cases H^s and H are identical.

(21) Taking Narora as standard the following formula has been evolved, based on the theory that a proportion of $4c$ is a proper width of a weir of 13 feet in height, and for greater or less heights the length is subject to variation in proportion to the square root of the heights of shutters above floor—or as $\sqrt{H^s} : \sqrt{13}$.

W is the width of floor or apron. Then

$$W = 4c \times \sqrt{\frac{H^s}{13}} \qquad (1)$$

In the sloping apron type B (*vide post*) H is taken for H^s.

The use of c as a factor makes the width some function of the quality of the sand, which is admittedly a sound proposition.

(22) In designing a weir the dimensions relating to l should for convenience sake be all multiples of the coefficient c as by this means the neutralisation of head at any point is at once ascertained each length in feet equal to c, representing the neutralisation of 1 foot of head

(23) In the present instance to revert to the design Fig 3 the width of the fore apron will be according to formula (1) $4c\sqrt{\frac{13}{13}} = 4c = 4 \times 15 =$ 60 feet This will leave $13 - 4 = 9c$ to be disposed of in the rear apron and in the sheet piling If the latter be given a total depth of $2c = 30$ feet for two lines of curtains a further item of $4c$ can be deducted leaving $5c$ for the width of the rear apron This width is measured backwards from the toe of the drop wall and is 75 feet Out of the 30 feet allowed to the sheet piling a depth of 15 feet will be placed below the weir wall equivalent to an effective length of $2c$ The remainder will be situated at the toe of the fore apron where it will serve another purpose in addition to its ordinary function viz of forming a protective barrier at the end of the masonry floor in case the loose stone continuation is washed out

Thus the full required value of l or of cH i e 195 feet is provided It may be noted that the small vertical drop due to the thickness of the rear apron as well as that due to the thickness of the toe of the fore apron has been neglected It will be seen that there are two steps in the gradient line (which is 1 in 15) one of 2 feet at the incidence of the rear sheet piling and another also of 2 feet at the termination of the floor The designer can of course vary this arrangement at pleasure by reducing the vertical curtains and increasing the rear apron

(24) As regards the thickness of the parts the rear apron is composed of clay puddle overlaid with stone riprap a construction which provides a heavy impervious platform economical in point of cost but not as fool proof as if constructed of masonry * It is given a thickness of 5 feet

We next come to the drop wall provided with crest shutters 3 feet deep which collapse automatically when freshets come down Its base thickness is made 10 feet or

$$\frac{H^c + d}{\sqrt{\rho}} \qquad (2)$$

d being that depth of film passing over the crest which is reciprocal to a down stream water level at the crest line d in this case is calculated to be $3\frac{1}{2}$ feet The $\sqrt{\rho}$ in this case is $\frac{4}{3}$ as ρ is taken for the brick wall at a value of 1 8 This formula of $(H + d) \div \sqrt{\rho}$ applies to all dwarf submerged weirs (*vide* par (42) Chap II) H^a is shutter crest above floor H is the head H^b weir wall crest above L W L H^c is the height of the masonry drop wall above floor

* The accident to Narora Weir in March 1898 resulted from the scouring away of the apron by currents parallel or diagonal to the weir —ED

(25) We now come to thickness of the fore apron at its root or commencement. This is really the most critical point in the whole design. This thickness can be varied at pleasure by the values given to the rear apron and to the rear sheet piling, and is a matter always kept in mind when deciding what these values should be. This thickness must not, as a rule, be less than 4 or 5 feet, except in very low weirs, and should not much exceed the higher dimension either, as otherwise the fore apron will become unnecessarily costly. Up to this point the head (h) neutralised by the length of creep is equivalent to that of 7c or 7 feet of head; this leaves a value of $(H - h)$ = 6 feet of upward water pressure. We have already noted that the effective weight must exceed that of the hydrostatic pressure by at least one-third. The floor is immersed and its specific gravity considered as reduced by unity; consequently the thickness of the floor (t) at this point can be expressed in the following formula:

$$t^{*} = \frac{4}{3}\,\frac{H - h}{(\rho - 1) \text{ or } \rho} \qquad (3)$$

$= \frac{4}{3}\cdot\frac{6}{1\frac{1}{4}} = 6.4$ feet. In (3), if the floor is immersed the denominator will be $(\rho - 1)$; if not, it will be ρ. It has been made 6¼ feet thick, tapering to 3¾ feet at the end—average 5 feet. The pressure trapezoid is shown in the space enclosed between the surface of the floor and the grade line. The terminal thickness of the floor, 3¾ feet, may be considered almost as a minimum. The insertion of the 15 feet deep sheet piling here causes 2 feet hydrostatic pressure at the toe of the apron. There is no objection to this, as the apron is capable of withstanding 3¾ feet pressure. In fact the arrangement may be considered as utilising a part at least of the excess weight at this point. Weight is, however, desirable, solidity being required in the fore apron apart from purely hydrostatical considerations.

(26) There now only remains the packed stone pitching or riprap which forms the talus of the weir body. The width of this is dependent on the same kinetic considerations which influence the width of the masonry floor, but with two others in addition. These are, firstly, the nature of the river bed, which will, as before, be represented in the formula by its coefficient; secondly, H^{b}, the height of the permanent obstruction above L.W.L. Where the floor surface is at L.W.L., H^{b} will equal H; where, however, it is raised as in Figs. 5 and 6, or where in other types there is no floor but an inclined apron, it will be the height of the obstruction above L.W.L. or normal bed level, which may be higher. Thus where no crest shutters are used H^{b} will equal H. The second matter requiring consideration is the flood discharge per foot run of weir, which is symbolised by the letter q. This item varies considerably, from 66 second feet over the Dehri Weir (Fig. 17) to 244 second feet over the Madaya (Fig. 22), and must necessarily have considerable influence on the width of the talus. Narora Weir (Fig. 4) will again be

* Bearing in mind, however, that brick masonry or concrete when saturated with water is increased in weight thereby to the extent of say [illegible]0 per cent, bearing in mind also the transverse strength of such material, the factor (4/3) might reasonably be omitted from formula (3).—Ed.

taken as a guide in framing a formula generally suitable for all conditions The end of the talus measured from the drop wall was originally 170 feet It is however, deemed that a width of 150 feet or of 10c would fairly represent a proper average value for (L) or the distance of the toe of the talus from the drop wall

(27) The extreme width of the apron including talus is clearly independent of that appropriated to the floor or masonry apron its width L is

TABLE I —FORMULA $L = 10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$ SHOWING ACTUAL AND CALCULATED VALUES OF L OR FLOOR AND TALUS WIDTH

River	Name of Work	Type	C	H^b	q	Calculated value of L	Actual value of L
Ganges	Narora	A	15	10	56	200	170
Colerun	Colerun	A	12	4½	100	106	72
Vellar	Pelandorai	A	9	11	140	180	101
Tampraparni	Srivakantham	A	12	6	90	102	105
Chenab	Khanki	B	15	11 5	150	275	110 to 170
Jhelum	Jhelum	B	15	11 5	150	225	166
Pennér	*Adimapali*	*B*	*12*	*8½*	*184*	*172*	*184*
	Vellore	B	12	9	300	228	232
	Sangam	B	12	8	147	151	145
Godaveri	Dauleshwaram	B	12	13	100	158	217
Jumna	Okhla	C	15	10	60	140	250
Kistna	Beswada	C	12	13	216	233	257
Sôn	Dehri	C	12	8	66	100	96
Mahanadi	Jobra	C	12	10	140	163	143
Madaya	Madaya	C	12	10	280	180	235
East Nara	Jamrao	B	15	7	10	43	110

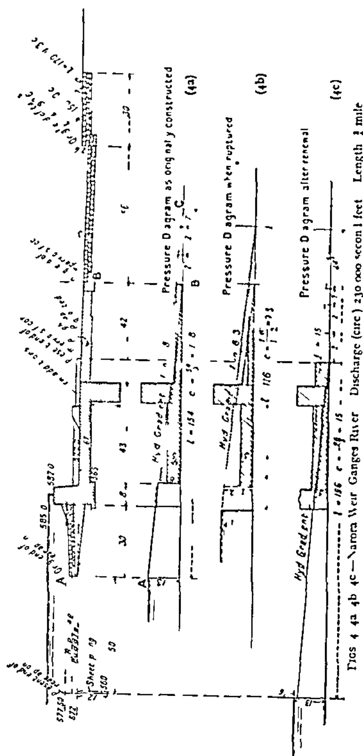
Type A Vertical drop wall with horizontal floor B Sloping masonry apron C Rock fill anicut with sloping apron

consequently measured from the drop or crest wall The formula will then become—

$$L = 10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}} \qquad (4)$$

For sloping aprons a somewhat higher factor viz 10½ or 11c might be adopted

This formula is grounded on the theory that the width L varies with the square root of the height of the obstruction (H^b) and with that of the unit flood discharge (q) the standard being what these values viz 10 and 75 respectively are in Narora Weir This height H^b as already stated is equal to H^c when there are no crest shutters and L W L is at floor level and is always the depth of L W L below the permanent masonry crest of the weir This



Figs 4 4a 4b 4c—Narora Weir Ganges River Discharge (circ) 230 000 second feet Length 1 mile

formula though more or less empirical gives results remarkably in consonance with actual values, and consequently will, it is believed form a valuable guide to the designer The table on p 155 shows data of existing works compared with the reckoning by formula (4)

(28) The Narora Weir itself forms a most instructive object lesson of the proper base length, or value of l suitable for sands of class 2

Fig 4 (full lines) represents the work as originally constructed The pressure diagram Fig 4a, shows that the hydraulic gradient works out to less than 1 in 12 and in addition to this the floor is very deficient in weight The hydrostatic pressure at the toe of the drop wall is 8 feet of water, to meet this the floor has a value of tp of only 5 feet The specific gravity is taken at 2 which figure the floor would not be likely to exceed being mostly formed of brick concrete In consequence of submergence the effective specific gravity is only unity This work stood as constructed with but little repairs for about twenty years,

when the rear apron of puddle protected by stone riprap was washed away. The diagram, Fig. 4b, clearly shows the excessive pressure in the floor, which is 12 3 feet of water up stream * of the weir

(29) In restoring the work, the rear apron was run out backwards, as shown dotted in Fig 4, to a distance of 80 feet beyond the drop wall, and was made $5\frac{1}{2}$ feet thick, of puddle covered with riprap and provided near the weir wall with a solid masonry covering The puddle foundation also was sloped down to the level of the floor base, so as to form a sound connection with the drop wall At its extreme termination sheet piling was driven to a depth of 12 feet below floor level, *i e*, below R L 572 The grouted pitching in the fore apron was relaid dry, the first 10 feet only having been rebuilt in mortar, so as to form a continuation of the impervious floor This had the effect of reducing the pressure on the floor A water cushion, 2 feet deep, was formed over the floor, by building a dwarf wall of concrete (shown on the section) right along its edge This adds 2 feet to the effective value of t_{ρ}, and is given credit for in the diagram Fig 4c † In this diagram it will be seen that the hydraulic gradient now works out at 1 in 15, the value of c which has been adopted for similar light sands, and from which that of others, as classes 1 and 3 have been deduced

(30) Some brief explanation will now be given of the diagram and the graphical method of construction

Take Fig 4 The first step is to ascertain what is the hydraulic gradient of the section as it stands The first incidence of the water pressure is at the point A From this starting point the horizontal length or width of the impervious rear and fore aprons up to B is 123 feet To this must be added the vertical components of l, which are 7 feet at the base of the drop wall and 24 feet or twice the depth of the curtain wall, at the termination of the floor proper This makes the total value of l to be $123 + 31 = 154$ feet

This is then measured out from A to C in Fig 4a The line AC then drawn is the hydraulic gradient of $\frac{154}{13}$, or 1 in 11 8 Now from C the vertical components are set out backwards in reverse order, thus the 7 feet depth is first marked, then the two 12 feet lengths, this last measurement coinciding with the point B

The outline of the area of water pressure is traced by drawing lines parallel to the hydraulic gradient AC from these several points till they intersect the verticals through the positions to which they respectively belong Thus, from A the hydraulic gradient continues unaffected until the drop wall is reached, here a step occurs The next line is drawn from the 7-foot point near C, parallel to AC This line continues till the curtain wall is reached, here another step down occurs, fixed by a parallel drawn up from the end of the 12 feet measured at the other side of the curtain, another step

* Pressure gauges inserted in the floor, showed that on March 26th 1898 the hydrostatic

takes place by the intersection of the last parallel line which continues down meeting the base at the termination of the impervious apron The area enclosed between the stepped sloping lines and the horizontal base evidently represents that of the water pressure acting below the base of the fore apron Over this the area of resistance (hatched closely) is now drawn the depths being $t\ (\rho - 1)$ or the actual thickness multiplied by the specific gravity reduced by unity the floor being below L W L and consequently immersed

In this the value of ρ for the floor proper is taken at 2 while that of the rough grouted block kunkur pitching at 1? The depth $t\rho$ is therefore equal to the actual thickness of the floor in the first case but is less in the grouted portion

The unbalanced water pressure can now be seen at a glance

The other diagrams are all constructed on the same lines Comparison between them is most instructive as the weak points in the original section and the means whereby the deficiencies were made good are at once demonstrated to the eye

We shall proceed to further analyse some other sections pointing out deficiencies suggesting improvements and occasionally producing an alternative design embodying the above

(31) The section of the Burra Weir Fig 5 offers some points of interest It is on the Mahanuddee system of which the Jobra Weir Fig 18 is the principal head work the value of c is therefore 12 The floor was originally built shorter than it is at present but the talus was washed away and had to be renewed with a prolongation of the floor itself This is an example of an overfall weir type A with the floor surface not at L W L as in the last case but raised above it to the extent of its thickness so as to avoid wet construction and is termed type A² In all such cases the proportionate hydrostatic stress on the floor is greater when the latter is just submerged than it is with the higher pressure

The head measured from summit level A which is also crest level to L W L (B) is 12 feet This will require a value of l or of cH of $12 \times 12 =$ 144 feet In the pressure diagram Fig 5a the actual value is only 112 feet giving an hydraulic gradient of 1 in 9 3

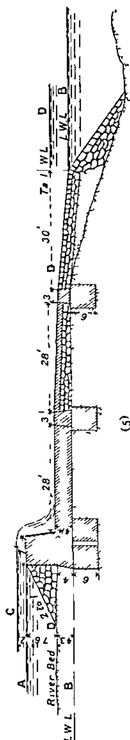
When the floor is submerged the tail water to effect this must rise 4½ feet to D the reciprocal depth of film over crest or d to produce this rise is estimated to be about 2 feet The summit level will then be at C The pressure diagrams of both cases are shown in Figs 5a and 5b In 5a the balance pressure on the floor near the toe of the weir wall is $t\rho - (H - h)$ $= (9\ 7 - 9) = +\ 7$

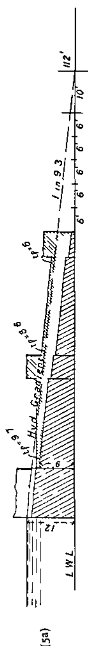
In Fig 5b the balance pressure at the same point is $5\ 4 - 6 = -\ 6$ feet At the end of the floor in Fig 5a it is $6 - (1\ 3) = +\ 4\ 7$ feet and in 5b is $3 - (1\ 1) = +\ 1\ 9$ feet only

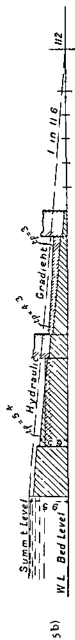
Thus we see that the floor is strained more with the lower than with the greater head On the other hand with the lower head Fig 5b the hydraulic gradient is flatter being 1 in 11 6 to 1 in 9 3 of 5a

(32) In all cases where the floor is founded above L W L it is necessary to test the thickness by formula (3) for another value of H which occurs

when the floor is under water This may be less than before, but may have a greater effect owing to the submergence of the floor.







Figs 5 5a 5b—Burra Weir

The greatest difference of levels always occurs when the summit level is at crest level and the tail channel empty, and this maximum possible

statical head is the value of H from which the length of creep l is calculated In this design the floor requires thickening at its root and widening or else has to be provided with deep vertical curtain walls According to formula (1) par (21) the width with $c = 12$ should be $48\sqrt{7} = 40$ feet, nearly it is actually 56 *feet although the last bay is not all formed* of solid masonry In remodelling a section of this kind a rear apron should first be provided concrete sheet piling may preferably be substituted for the under sunk blocks

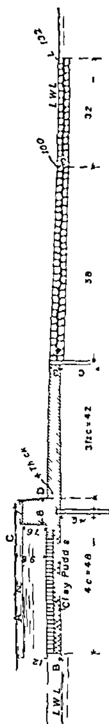


Fig 6—Alternative Section on Burra Weir

(33) Fig 6 is the profile remodelled according to the author's views The fore apron is made $3\frac{1}{2}c$ wide, or 42 feet A row of piling of a depth of 12 feet ($= c$) will be placed at the end of the apron This is equivalent to a length of creep of $2c$ Consequently the fore apron takes altogether $5\frac{1}{2}c$ in length leaving $12 - 5\frac{1}{2} = 6\frac{1}{2}c$ for the rear apron and rear sheet piling which will be made $4c$ and $2\frac{1}{2}c$ respectively The thickness of the apron at the toe of the weir wall with the full head of AB will be according to formula (3) par (25) $\frac{4}{3} \times \frac{12 - 6.5}{2} = 3.7$ feet The same calculated from formula (3) *with a reduced head of CD or of 9.5 feet* will be $\frac{4}{3} \times \frac{9.5 - 6.5}{1} = 4$ feet The latter dimensions will thus be adapted The apron will taper to a thickness of 3 feet at its extremity and will be horizontal at the base inclined on the surface

The talus will slope down to L W L tapering from 4 to 3 feet thickness The width of the talus calculated by formula (4) par (27) will be $10c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}}$ which works to $120 \times 1.1 = 132$ feet in which $H^b = 12$ feet $c = 12$ and q is taken as 75 second feet

The revised section would it is believed cost less than the original The raising of the floor above L W L although it undoubtedly usurps the waterway to a certain extent results in considerable economy If the floor surface were placed lower it would have to be not only thicker but longer and the construction being below spring level would be more costly This type of raised floor will as already noted be classed as a sub type of A viz as type A_2

(34) Fig 7 is of a low weir of type A on the Upper Colerun River in the Madras Presidency The original work was also built with too short a floor, subsequently lengthened to its present dimensions This shows that these works were designed haphazard in fact, as far as is known rules for guidance in design based on definite principles are now set out for the first time

The principle of length of percolation influencing design is not the author s invention but its practical application has never hitherto been rendered available by the promulgation of formulas suitable to the varying conditions met with in practice

The Colerun Weir has lately been entirely rebuilt in rear of the present site, and is changed to an open regulator with 25 feet spans and roller counterbalanced gates operated by gear the present floor forming part of the new work It is believed that the head of water has also been considerably increased A photograph of the piers and gates is given in The

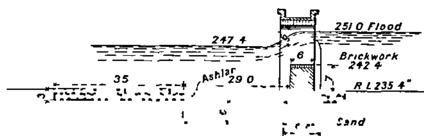


FIG 7 —Colerun Weir Discharge 284 000 second feet Length 2 926 feet

Irrigation Works of India (Buckley) which is decidedly useful but a working drawing if even a rough diagram, would be much more so

(35) The next three examples will be of another type designated type B In these weirs there is no direct vertical drop the fore apron not being horizontal but sloping down from the crest to the L W L or to a little below it, the talus beyond being either also on a flat slope or else horizontal

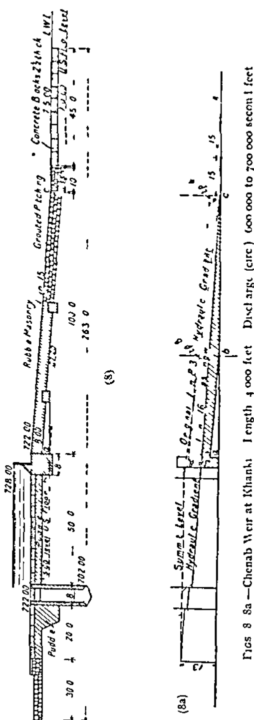
In the modern examples of this type which will first be examined the height of the permanent masonry weir wall is greatly reduced with the object of offering as little obstruction as possible to the passage of flood water

The canal summit level is attained by means of deep crest shutters In the Chenab Weir (Fig 8), the weir or rather crest wall is 7 feet * high above L W L , while the shutters are 6 feet high it therefore holds up 13 feet of water, much the same as was the case with Narora Weir

The object of adopting the sloping apron is lightness and cheapness of construction and reduction of hydrostatic upward pressure The dis advantage of this type lies in the contraction of the waterway below the breast wall which causes the velocity of overfall to be continued well past the crest With a direct overfall on the other hand a depth of 7 feet for

* Increased to 11 5 feet in the year 1918 or to 17 5 feet up to top of shutters

water to churn in would be available at this point, this checks the flow and the increased area of the waterway rendered available must reduce the velocity. For this reason although the action on the apron is possibly less that on the talus and river bed beyond must be greater in type B than in the drop wall A type.



Figs 8 8a —Chenab Weir at Khanki Length 4 000 feet Discharge (circ) 600 000 to 700 000 second feet

(36) This work, like Narora failed for want* of sufficient effective base length and it consequently forms a valuable object lesson.

As originally designed only a short rear apron sloping down up stream at 1 in 3 of stone behind the crest wall was provided. The value of l up to the termination of the grouted pitching was but 108 feet, whereas it should have been $c \times H$, or $15 \times 13 = 195$ feet. The hydraulic gradient as shown in Fig 8a, was only 1 in 8.3.† This neglects the small vertical component at the breast wall. In spite of this deficiency of effective base width, the floor did not give way for some years until gradually increased piping beneath the base caused its collapse.

Owing to the raised position of the apron it is not subject to high hydrostatic pressure. At its commencement it is 10 feet below the summit level and 9 feet of water acts at this critical point, this is met by 4 feet of masonry insubmerged of specific gravity 2, which almost balances it. Thus the apron did not blow up, as was the case with Narora, but collapsed.

* But see Editor's Note (O) at end of this Chapter —Ed

† But at the time of the accident the water level up stream of weir was only 7 6 0 and down stream 72 which indicates a head of only 4 feet. The water stood only 4 feet deep on weir crest too low a head to have caused piping under a crest wall 8 feet deep and under a masonry floor 4 feet thick and 60 feet long and sloped at 1 in 15. The wall failed during construction and again in the year following completion. See Editor's Notes at end of this Chapter —Ed

(37) Some explanation of the graphical pressure diagram Fig 8a is required, as it offers some peculiarities differing from the last examples The full head, or H, is 13 feet Owing however, to the raised and sloping position of the apron, the base line of the pressure area will not be horizontal, coinciding with the L W L, but will be an inclined line—from the commencement a as far as the point b, where the sloping base coincides with L W L, from b onward, the base will be horizontal With a sloping apron the pressure is pretty well uniform, so that the water pressure area is not wedge-shaped but approximates to a rectangle, the apron therefore is also properly rectangular in profile, whereas in the overfall type the profile is, or should be, that of a truncated wedge

(38) After the failure of this work, the restoration was on very similar lines to that at Narora An impervious rear apron 70 feet long was con-

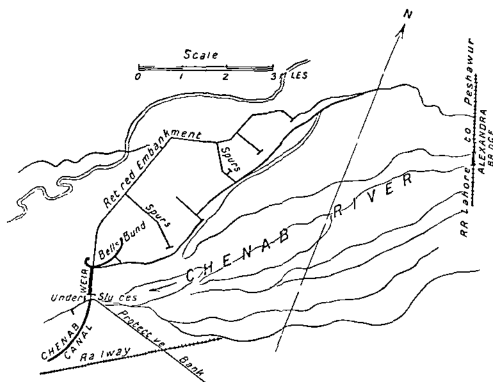
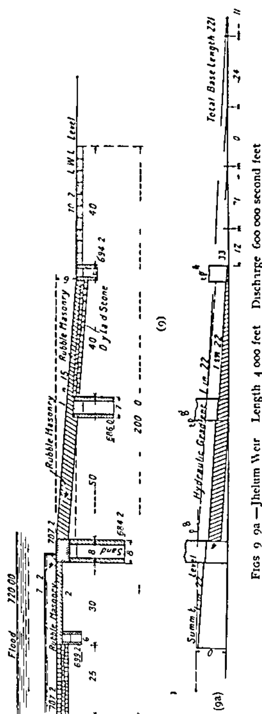


Fig 8b—Sketch Plan of Khanki Head Works

structed of puddle covered with concrete slabs, grouted in the joints, a rear curtain consisting of a line of rectangular undersunk blocks 20 feet deep was provided These additions reduce the gradient 1 in 16 A site plan of the weir is given in Fig 8b

(39) The Jhelum Weir (Figs 9 and 9a) is another example of type B In this case the hydraulic gradient is now 1 in 22, affording large superfluity of stability If all the lines of curtain walls were abolished and one line of

sheet piling below the breast wall of a depth of 15 feet substituted, the hydraulic gradient would be reduced to a more reasonable ratio of 1 in 16



Figs 9 9a —Jhelum Weir Length 4 000 feet Discharge 600 000 second feet

The value of L, or distance of toe of talus from the crest wall, measures 135 * feet According to formula (1) $c = 15$, $H^b = 6$, and $q = 135$ It should be therefore

$$10 \times 15 \times \sqrt{\frac{6}{10}} \times \sqrt{\frac{135}{75}} = 160$$

feet, which is not a very close correspondence A site plan of the head works is given in Fig 9b which is most instructive

(40) Another instance of the same type is that of the Jamrao Weir (Fig 10) The head here is 7 feet, requiring a base length, or value of l, of $7 \times 15 = 105$ feet, it is actually 167 feet, excluding the wooden sheet piling in the rear This gives a gradient of 1 in 24, which is in excess of requirements The diagram in Fig 10a shows the effective floor weight at the critical point of the apron, which is here beyond the second inset, to be 6 feet, being 3 feet of brickwork at specific gravity 2 insubmerged The water pressure is 4 75 requiring one-third excess, viz, $6\frac{1}{3}$ feet From the point where the surface slope of the apron intersects the L W L, the whole floor is submerged and consequently tp is only 3 feet at which value it remains until the fore curtain is reached If the tail water rose to weir crest level and the shutters were still up (a possible contin-

* This figure should be 165 including a talus of stone 30 feet long at the tail end And q should be 150 which makes the calculation $10 \times 15 \times \sqrt{6} \times \sqrt{\frac{150}{75}} = 168$ feet In the year 1918 the weir shutters were dismantled and the masonry crest was built up to 711 5 See also Editor s Notes at end of this Chapter —Ed

gency), the head acting below the apron would be $4 - 3\frac{1}{4} = \frac{3}{4}$ foot only With the crest shutters down the depth d above crest would not exceed 2 feet

Material for loose stone filling was not obtainable at this site so that the

FIG 9b —Head Works of Jhelum Canal

rear apron had also to be built of brickwork and the talus constructed of concrete blocks

The width of the masonry apron according to formula (1) should be $4c\sqrt{\frac{H^a}{13}}$ Here $H^a = H = 7$ and W works out to $4 \times 15 \times 75 = 45$ feet

(10)

(10a)

FIGS 10 10a —Jamrao Weir Length 1 250 feet

It is actually 70 feet The width of talus from crest wall (formula (4)) should be $11c \times \sqrt{\frac{7}{10}} \times \sqrt{\frac{10}{75}} - 43$ feet it is actually 110 feet

(41) Another example of the same type is furnished in Fig 11 of the Adimapali Anicut This work which is of an older pattern is not provided

with the usual collapsible crest shutters Under such conditions the hydrostatic stress on the underside of the apron has generally a greater effect when the down stream water level is at crest level than when the channel is empty although the head is less This is accounted for by the reduction of effective weight owing to submergence The hydraulic gradient works out to 1 in 12 6 if half value only is accorded to the circular well curtains which are not perfectly watertight When the tail channel is empty the head (*H*) on which the value of *c* must always be calculated is 8½ feet but as the root of the sloping apron is situated at 4 feet above L W L the head acting on the base there is but 4 5 feet This amount is further diminished by the loss of head (*H*) due to percolation or by the length of creep up to this point divided by *c* which amounts to $\frac{14}{12\ 6} = 1\ 1$ foot leaving a balance of 3 4 feet The pressure is met by the resistance of $t\rho$ or by $4\ 5 \times 2 = 9$ feet the difference being $9 - 3\ 4 = 5\ 6$ feet in favour of stability

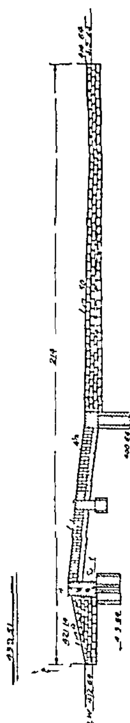


Fig 11 —Admapall Weir Penner River Length 2 43 feet Discharge 413 500 second feet

This work is situated high up on the Penner River and is subject to very heavy floods the unit foot discharge of which amounts to 184 second feet a quantity 2½ times greater than that of the Ganges at Narora These abnormal conditions will necessarily affect the design in the increased width of apron and talus

The width of the fore apron is 60 feet According to formula (1) it comes to 45 feet This discrepancy is easily accounted for by the abnormal conditions prevailing With regard to the talus formula (4) makes allowance for high unit flood discharge According to this formula taking 10½ and not 10 as the multiplier of *c* par (37) $L = 10\frac{1}{2}c \times \sqrt{\frac{8\ 5}{10}} \times \sqrt{\frac{184}{75}} = 180$ feet it is actually 184 feet

(42) The previous examples of types A and B have all been cases where the weir has an appendage an impervious floor which is subject to hydrostatic pressure There is another very common type which will be termed C in which there is no impervious floor and the material which composes the body of the weir is not solid masonry but a porous mass of loose stone filling the

only impervious parts being vertical, not horizontal walls. In spite of this apparent contrariety it will be found that the same principle, viz., that of length of enforced percolation, influences the design in this type *in the same way as in the others we have been considering.*

Fig. 12 represents a wall upholding water to its crest, and resting on a previous substratum as sand, gravel, or boulders, or a mixture of all three

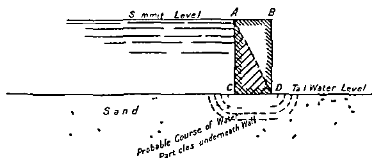


Fig. 12

materials. The hydraulic gradient is AD, the upward pressure area, ACD, and the base CD, is the line of percolation. Unless this base length is equal to AC multiplied by the coefficient obtained by experiment, piping will set in and the wall will be undermined.

Now, as shown in Fig. 13, let a mass of loose stone be deposited below the wall. The weight of this stone will evidently have an appreciable effect in checking the disintegration and removal of the sand foundation. The water will not have a free untrammelled egress at D; it will, on the con-

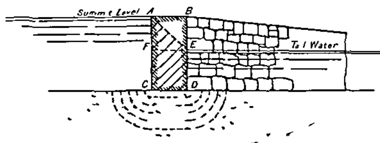


Fig. 13

trary, be compelled to rise in the interstices of the mass to a certain height, FE, which is determined by the extent of the obstruction caused to the flow.

The resulting hydraulic will now be* AE, flatter than AD but still too steep for permanency.

(43) In Fig. 14 the wall is shown backed by a rear apron of loose stone and the body extending to F. The water has now to filter through the rear

* It should be noted, however, that the author has assumed a much higher level of tail water in Fig. 13 than in Fig. 12, which materially affects the comparison.—Ed.

apron underneath the wall and up through the stone filling in the fore apron. During this process a certain amount of sand will be washed up into the porous weir body, and the loose stone will sink, until the combined stone and sand forms a compact mass offering an obstruction to the passage of the percolating water much as, or more than exists in the sand itself, and possessed of far greater resisting power to disintegration This will cause the level of water at E to rise until equilibrium results Where this is the case, the hydraulic gradient is flattened out to some point G near I If a sufficiently long body is provided the resulting gradient will be equal to that found by experiment to produce permanent equilibrium The mass after the sinking process has finished, is then made good up to the original profile, by fresh stone filling Near F, the toe of the slope, the stone offers but little resistance either by its weight or depth, so it is evident that the slope of the prism should be flatter than that of the hydraulic gradient

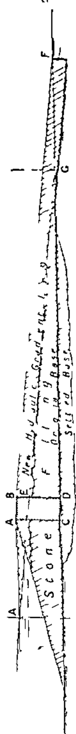


Fig 14

The same action takes place with the rear apron, *which soon becomes silt-charged so as to be impervious*, or nearly so, to the passage of water, but until silt is deposited in the river bed behind, as will eventually occur right up to crest level, the thin portion of the rear slope as well as the similar portion of the fore slope near the toe cannot be counted as effective Consequently, of the whole base length CF, roughly about one-quarter can be deemed inefficient as regards length of percolation As the consolidated lower part of the body of the weir gains in consistency, it can well be subject to hydrostatic pressure Consequently the value of t_p of the mass should be in excess of that of $(H - h)$, just as was the case with an impervious floor

(44) In Fig 15 a further development is effected by the introduction of vertical body walls of masonry in the pervious mass of the fore apron These impervious obstructions materially assist the stability of the foundation, so much so that the process of underscour and settlement, which must precede the equilibrating of the opposing forces in the purely loose stone mass, need not occur at all, or nothing like to the same extent If the party walls are properly spaced, the surface slope can be that of the hydraulic gradient itself and thus ensure equilibrium This is clearly illustrated in Fig 15 The water passing underneath the wall base CD, will rise up to the level F, the point E being somewhat higher, and similarly percolating under the other

walls through the sand substratum, will fill up all the partitions with water. The head AC will therefore be split up into four steps

The value of the stanchness to percolation of the rear apron is so marked that it should be rendered impervious by a thick underlayer of clay, and not left to more or less imperfect surface silt stanching as has hitherto been the case The specific gravity of stone filling with 50 per cent void is about 1 3 When immersed, this will be reduced to 8, for the following reason the specific gravity of the solid stone is 2 6, when immersed, 1 6 Divide this by 2 as half is void the result is 8 In cases where it is convenient to assume the full head acting on the whole work, the weight of material, plus that of the water filling the interstices, will represent the actual overlying weight This can be considered as having a specific gravity of $8 + 5 = 1\ 3$

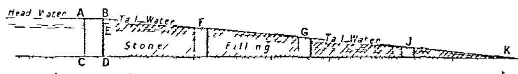


FIG 15

This happens to be identical with that of the unsubmerged mass, which is but a coincidence.

(45) Fig 16 is the section of the Okhla Weir over the Jumna River situated eight miles below the historic city of Delhi It is remarkable as being the first rock fill weir not provided with any lines of curtain walls projecting below the base line, which adjunct had hitherto invariably been adopted The stability of its sand foundation is consequently entirely dependent on its weight and effective base length As will be seen, the section is provided with three body walls in addition to the breast wall The slope of the fore apron is 1 in 20 It is believed that a slope of 1 in 15 would be equally effective, a horizontal talus making up the continuation, as has been done in the Madaya Weir (Fig 22, *post*)

The head, when the shutters are up and the weir body empty of water, a condition that could hardly exist, is 13 feet This would require an effective base length, or l, of 195 feet, the actual is 250 feet, but, as noticed previously, the end parts of the slopes cannot be included as effective, consequently the hydraulic gradient will not be far off from 1 in 15 The weight of the stone, or $t\rho$, exactly balances this head at the beginning of the fore apron, as it is $10 \times 1\ 3 = 13$ feet If the water were at crest level and the weir full of water, $t\rho$ would $= 8$ feet, or, rather, a trifle less, owing to the lower level of the crest of first body wall This head of 13 feet is broken up into four steps, the first, 3 feet deep, acting on a part of the rear apron together with 30 feet of the fore apron, say, 1 in 15, the rest are 1 in 20 A slope of 1 in 15 from the first party wall would cut the base at a point 40 feet short of the toe Theoretically a fourth party wall is required at this point, but practically the riprap below the third dwarf wall is so stanched

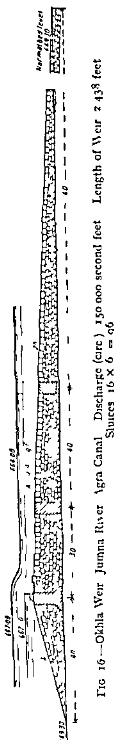


Fig 16—Okhla Weir Jumna River Agra Canal Discharge (circ) 150 000 second feet Length of Weir 2 438 feet
Sluices 16 × 6 = 96

with sand as not to afford a free egress for the percolation, consequently the hydraulic slope may be assumed to continue on to its intersection with the base As already noted, material would be saved in the section by adopting a reliably stanch rear apron and reducing the fore slope to 1 in 15, with a horizontal continuation.

This type (C) is only economical where stone is abundant, it requires little skilled labour, or masonry work On the other hand the mass of the material used is very great, much greater, in fact, than is shown by the section This is owing to the constant sinking and renewal of the talus, which goes on for many years after the first construction of the weir.

The action of the flood on the talus is undoubtedly accentuated by the contraction of the waterway due to the high sloping apron The flood velocity 20 feet below the crest of Okhla Weir has been gauged as high as 18 feet per second

(46) Another typical example of this class is the Dehri Weir over the Son River (Fig 17) The value of l, if the apices of the two triangles of stone filling are deducted and the curtain walls included, comes to about 12 H, 12 being the coefficient adopted for this class of sand

The curtain walls, each 12,500 feet long, must have been enormously costly From the experience of Okhla, a contemporary work, on a much worse class of sand curtain protection is quite unnecessary if sufficient horizontal base width is provided The head on this weir is 10 feet and the height of breast wall 8 feet, tp is therefore, 1 3 × 8 = 10 4, which is sufficient, considering that the full head will not act here The lines of curtains could be safely dispensed with if the following alterations were made (1) Rear apron to be reliably stanched in order to throw back the incidence of pressure and increase the effective base length, (2) three more body walls to be introduced, slope 1 in 12

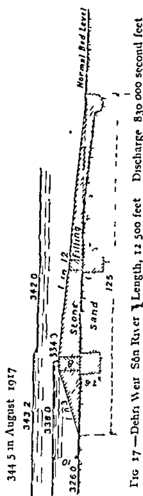


Fig 17—Dehri Weir Son River Length, 12 500 feet Discharge 830 000 second feet

retained but base to be dredged out towards apex to admit of no thickness under 5 feet This probably would not increase quantities except in masonry above what they are now and would entirely obviate nearly 5 miles of undersunk block curtain walls

(47) Figs 18 and 18a are of the weir at Jobra over the Mahanadi River also in Bengal The site plan is Fig 18b The head measured up from normal bed level viz R L 54 50 is 13 feet identically the same as in Okhla Weir The total base width is however 173 feet to 250 feet of the latter This great difference is due to the nature of the sand in the river The flood discharge per foot run is $\frac{900\ 000}{6\ 400}$ = 140 second feet The weir has two lines of curtain walls which add 34 feet to whatever is the effective base width With $c = 12$ the correct value of l will be $12 \times 13 = 156$ the actual is $173 + 34 = 207$ feet The base length is therefore sufficient

(48) Another example of a rock fill crib weir designed by the author for some intermittent torrents with deep sandy beds in upper Burma is given in Fig 19 This type was necessitated by the exigencies of the situation neither skilled labour nor lime for mortar being available and the time of construction was limited to a few months The object of the framing of posts and laggings is to act as a support for the covering boards These are essential to prevent the stone filling which was not packed on the surface from being washed out The foreslope is 1 in 15 formed by steps of 1 foot at 15 foot intervals An improvement would have been to have carried the longitudinal vertical

(18) (18a)

Figs 18 18a —Jobra Weir Mahanadi River Length 6 400 feet Discharge 900 000 second feet

planking at each step, right down to the sand foundation instead of stopping short, as was done They would then act the same way as the body walls shown in Figs 15, 16, etc, and be of great value in affording stability to the sand foundation Even without them these weirs answered well, the settlement not being great This was due to the rapid silting up of the river bed, the first freshet having piled sand up to crest level, thus rendering the weir watertight One of these weirs, whose foreslope was altered to 1 in 10, settled considerably and required renewal of the sunk

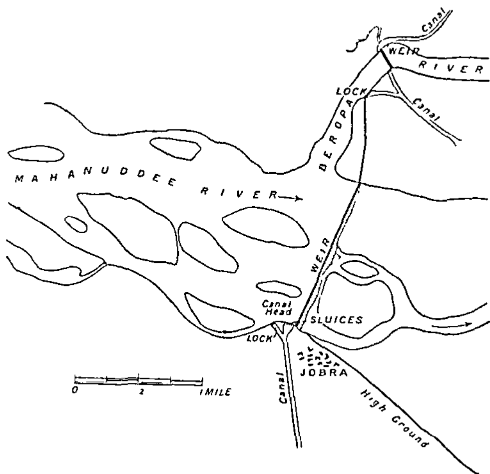


FIG 18b —Site Plan of Jobra Head Works.

stone-filling, which proved that the hydraulic gradient was too steep The body had to be lengthened in consequence A photograph of one of these weirs is given in Chap VIII, Fig 27

(49) In Fig 20 is an improved alternative section on the same lines, in which clay is used as a stratum below the stone-filling The base is therefore rendered impervious, and the correct value of l can be estimated with accuracy, and any subsequent settlement is rendered impossible It will probably also cost less The scantlings used are of the lightest possible

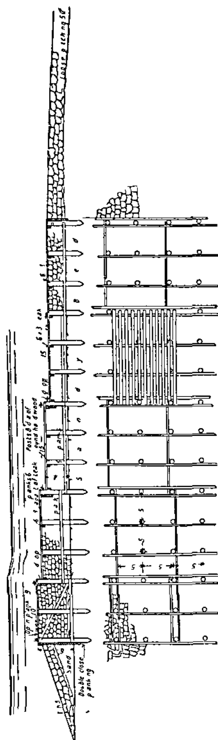


Fig 19 —Crib Weir in Burma

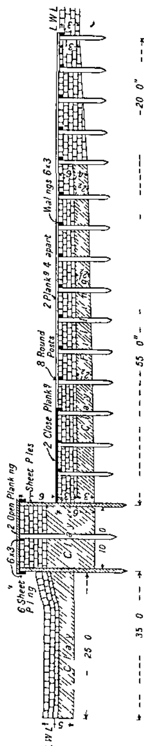


Fig 20 —Alternative Section for Crib Weir

dimensions, the laggings being 6 inches by 3 inches, the post 8 inches diameter round hard wood, and the planks were only 1 inch thick, these were spaced 6 inches apart and were screwed down with coach screws Heavy scantlings of light wood are unsuitable for crib work as they become buoyant

when immersed, thus reducing the weight of the weir body, so as often to require anchoring down, to prevent the whole being floated off

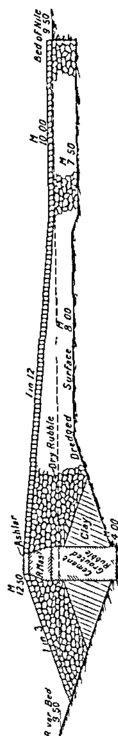


Fig 21 —Damietta Subsidiary Weir (Egypt)

(50) Fig 21 is a section of Damietta and Rosetta subsidiary weirs across the Nile River below the Grand Barrage Fig 21a is a site plan

The profile of this work is of type C, but the method of construction renders this work a most valuable object-lesson The work was carried out without any pumping, all material having been deposited in the water of

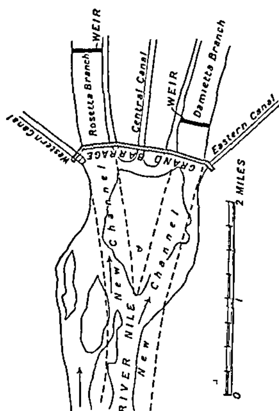


Fig 21a —Site Plan of Damietta and Rosetta Weirs

the Nile River First the profile of the base was dredged out, as shown in the section Then the core wall was constructed by first depositing loose stone in a temporary box or enclosure secured by a few piles from barges floated alongside The whole was then grouted with cement grout poured through pipes let into the mass On the completion of one section all the appurtenances were moved forward and another section built, and so on until the whole wall was completed Clay was deposited at the base of the core wall and the profile then made up by loose stone-filling

This system of subaqueous construction proved eminently satisfactory. Details of the construction of these two weirs are to be found in Sir Hanbury Brown's "Irrigation" (Constable), a most valuable addition to irrigation literature. Notwithstanding these excellent innovations in methods of rapid construction, the profile of the weir itself is open to the objection of being extravagantly bulky even for the type adopted, the base having been dredged out so deep as to greatly increase the mean depth of the stone filling.

It is open to question whether a row, or two rows of concrete sheet piles would not have been just as efficacious as the deep breast wall, and would certainly have been much less costly. The pure cement grouting was naturally very expensive, but the admixture of sand proved unsatisfactory, as the two materials of different specific gravity formed layers, and so pure cement had perforce to be used. It may be noted that the value of L here is much less than indicated by formula (4). At Narora Weir L is $11c$, or 165 feet long, here with a value of c of 18, L is but 150 feet $= 8\frac{1}{3}c$. The values of H^b are the same in both weirs.

It is a great advantage when time is short to have the mass of a large work like this constructed by unskilled labour which can be crowded into the work, collection of stone having been made a year previous. The provision of mortar and the building of concrete or masonry by skilled labour also involves a great deal of arrangement and supervision which is avoided when loose rock construction is adopted. A site plan is given in Fig. 21a.

(51) The section of the recently constructed Madaya Weir in Burma is shown in Fig. 22. On account of the exceptional heavy unit flood discharge, the conditions are quite abnormal so that although the material of the bed of the river is a shingle and coarse sand with value of c of about 10, yet the width of the talus exceeds even that of Okhla.

In the Madaya Weir the rear apron has been properly designed of impervious clay puddle, well protected on the surface and at its junction with the crest wall. This weir is only 200 feet long.

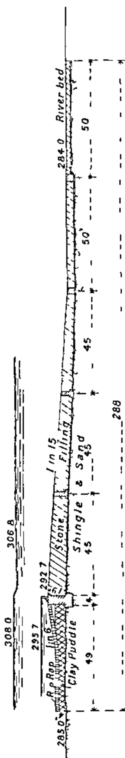


Fig. 22.—Madaya Weir (Burma)

(52) As an instructive example of what can be accomplished with simple clay filling in the body of a weir, with a mere covering of bricks laid on edge, the Sidnai Weir section (Fig. 23) is herewith given.

The weir is founded partly on deep sand and partly on clay The profile, where on clay, is similar to the figure but with the base cut off at the top of the piles Clay is heavier than stone filling having a specific gravity of 1 8 to 2 against 1 3 of the stone (with 50 per cent voids), consequently if its protection can be ensured it is a valuable material, having further the

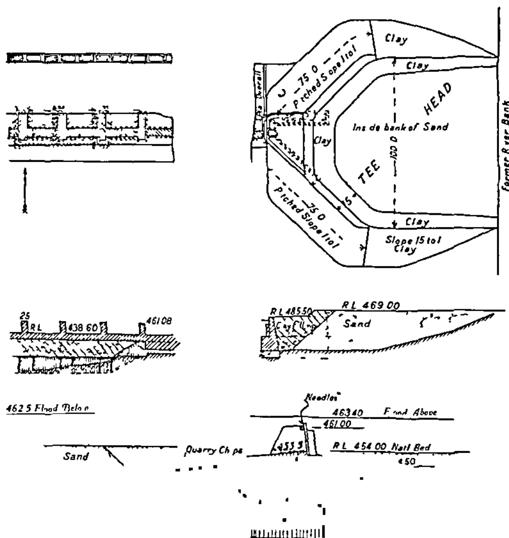


Fig 23 —Sidnai Canal Needle Weir Sutlej River Length 32 × 20 = 640′

advantage of being impervious It is considered that its use should not be confined to the rear apron of weirs but be also employed below rock filling, or concrete blocks or rubble masonry in the fore apron There is no reason why this should not be done

(53) In Fig 24 is an example of a 20 feet high overfall weir of type A on boulders and gravel It is of the Granite Reef diversion weir, Salt River project, of which the Roosevelt Dam is the main feature The profile,

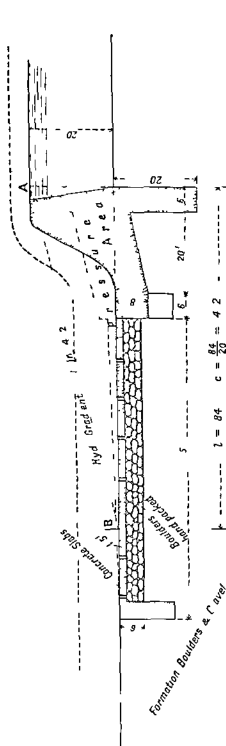


FIG 24 — Granite Reef Weir Salt River Arizona

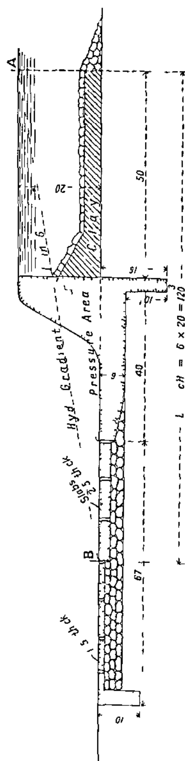


FIG 25 — Alternative Section

which has stood the test of actual trial, is extremely valuable as an object lesson wherefrom a reliable value for the coefficient c for river beds formed of boulders and gravel can be deduced. The estimated length of creep or

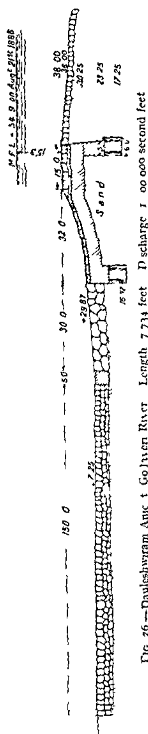


Fig 26—Dauleshwram Anicut Godaveri River Length 7734 feet Discharge 1 00 000 second feet

amounts to 84 feet which when divided by the head of 20 feet the quotient c is 4 2 The peculiarity of the section consists in the floor being almost entirely relieved of hydrostatic pressure by spaces having been purposely left between the 10 feet square concrete blocks which form the surfacing of the floor This arrangement, by shortening the base length, effects considerable reduction in the pressure area of the drop wall Such a device, however, would not be practicable when pure sand forms the foundation, as it would inevitably be blown up through the interstices and washed away, causing the floor to collapse The fore curtain is also pierced by vents with the evident object of reducing the hydrostatic pressure below the weir by allowing the percolating water a free passage This idea is purely chimerical the only possible effect of the vents will be to nullify the utility of the fore curtain as providing an additional length of creep In the diagram the outlet not being quite free a reduced pressure of $1\frac{1}{2}$ feet is allowed to exist at the commencement of the floor tapering to *nil* at its extremity

(54) In Fig 25 while retaining the general characteristics of the original profile the following modifications are suggested —

An impervious rear apron and the floor widened considerably The head of water acting on the back of the weir wall is reduced from 20 to 16 feet; the section can therefore be reduced The hydraulic gradient provided is 1 in 6, the previous one of 1 in 4 not being deemed flat enough The floor covering slabs have been increased in thickness from $1\frac{1}{2}$ to $2\frac{1}{2}$ feet, the previous thickness not providing sufficient weight as they are practically submerged and lose weight from displacement As thus altered the work will not cost more and be very much more stable The weir is 1 000 feet long The section over rock is given in Fig 31 par (58), Chap II

(55) A section of the Grand Anicut * at Dauleshwram over the Godaveri River is given in Fig 26 This work is remarkable as differing from the ordinary anicut type in many respects It was constructed some time in the forties of the

* For map see Editor s Notes at end of Chap XI —Ed

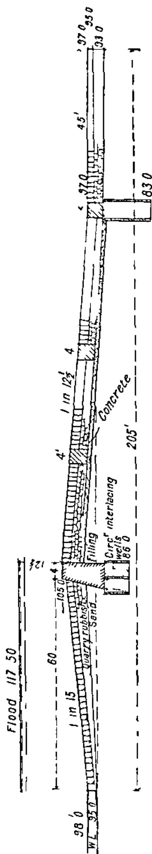


Fig 27 —Sangam Anicut Penner River Length 4 076 feet Discharge 600 000 cubic feet

Figs 28 28a —Beswada Anicut Kistna River Length 4 000 feet Discharge, 736 000 second feet.

nineteenth century, and it was deemed desirable, in the case of an overfall, to resolve the current in a horizontal direction

This weir consists of two breast walls founded on rows of circular 9 inch thick wells, sunk 6 feet in the sand and filled with rubble stone These breast walls are 36 feet apart and are connected by a wide horizontal crest and curved apron, bringing the level down to 8 feet below crest and to 2 feet above that of the normal river bed Beyond this masonry apron is packed pitching, 30 feet in length where a shallow masonry wall finishes the made apron The talus of more or less hazard stone pitching extends for another 150 feet The apron and horizontal crest are founded on sand filling, a system at once economical, and with this description of coarse sand quite safe This making up by sand filling is quite a common practice in Madras works which might well be followed with advantage in many cases where rubble filling has been resorted to

The width of the weir is 217 feet, which is exceptionally great, being only exceeded by the 250 feet of the Okhla Weir, which work, however, is founded on sand of a very inferior description to that in the Godaveri River, and by the Madaya Weir 235 feet The crest of this weir is 11 feet above normal bed level

(56) The Sangam Anicut (Fig 27) is a work on the same river as the Adimapali but lower down the stream, the whole discharge is greater, but the unit discharge is less being 147 to 184 of the latter (*vide* Table I, par (27)) Its fore apron which is impervious, is 60 feet long

(57) The Beswada Anicut, Figs 28 28a, and 28b, at the head of the Kistna River canal system which was constructed 1854-55, is a notable

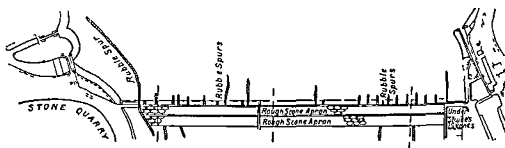


FIG 28b —Beswada Anicut Site Plan

work The river at the site of the work had actually a flood velocity of 10 second feet The weir obstructs half the flood water section

In the list given in Table I, par (27), the unit discharge exceeds that of any other work, excepting the Madaya and the Pennér at Vellore

A row of circular wells of the usual Madras type forms the foundation of the breast wall Where the bed was deep in the cold weather channel it was first filled up with sand above L W L, and the wells were then built on this bank and undersunk in the dry the current having been diverted elsewhere

The remaining part of the low channel was then filled up with stone This is explained in Figs 28 and 28a

In the case of the Mahanadi River Weir, at Jobra, a different procedure was adopted, as is shown in Fig 18a Where the deep bed existed it was filled up to above L.W L with stone, the line of blocks being entirely omitted at a part where, apparently, they are most wanted The breast wall was then built on the rubble mass

(58) The late General Rundall deserves the gratitude of posterity for having had the foresight to conceive, and the boldness to execute a large loose rock weir without any foundations below the bed of the river At the time Okhla Weir was under construction, most engineers, as the author well recollects, prophesied its utter failure

(59) The Laguna Weir (Fig 29) has the distinction of being the only weir of type C in America It is consequently termed the "Indian' weir

The Colorado River has all the attributes of one of the deltaic rivers of India, the main difference lying in its low flood level and comparatively small discharge, which dwindles down to a very small amount in the dry season

From the description of the sand, the coefficient will be at least 15 * The weir is 4,500 feet long, as measured on the plan, and the depth of afflux is 5 feet The crest of this weir is placed at high flood level, consequently it blocks the whole of the water section, a procedure which is quite unique

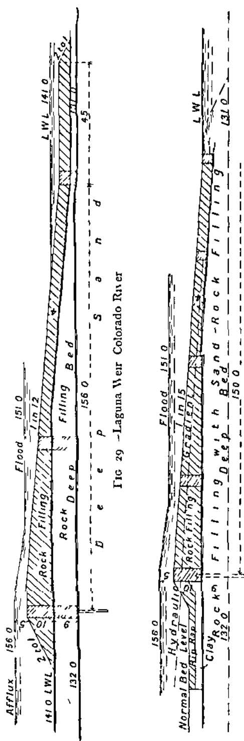


FIG 29 —Laguna Weir Colorado River

FIG 30 —Alternative Section

The unit discharge is from 35 to 40 second-feet only, the overfall being

* Practical analysis of samples seems desirable —ED

free No crest shutters are used, and the rear apron is not stanched with a clay layer. The object of crest shutters is to reduce the obstruction to floods, and consequently the scouring action of the overfall

(60) The breast and body walls are carried right down to the deep bed level, and the former has sheet piling below that again

The body walls are not considered to be sufficiently near to each other to be properly effective These suggested improvements are embodied in the alternative profile shown in Fig 30 The wooden sheeting is retained below the breast wall, but solely with the object of blocking the deep channel, which during construction should be diverted through the sluice channel

If this is done there is no reason whatever why the deep channel should not be filled up with nothing but sand, the latter being kept in place by a heap of rubble deposited above and below it *

Sand filling is commonly practised in Madras weirs and is effective, provided there is no current which will work it away, or its removal by scour is otherwise prevented

The plan of these head works is given in Fig 13, Chap VII, par (28)

(61) Fig 31 is a section of the Srivakantham Anicut over the Tampraparni River As their unpronounceable names suggest, these works are situated in the Madras Presidency Although the section seems insignificant, its length is a quarter of a mile, and the flood water of the river is 15 feet deep, the unit discharge is but 90 second-feet

This work is practically founded on hard clay The sand being quite shallow, the curtains fore and aft of the floor penetrate into the clay substratum The work is, therefore, free from hydrostatic pressure. If it were not for this the length of the floor would be quite inadequate

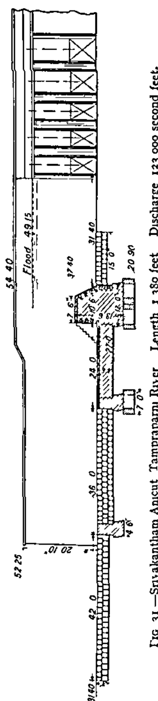


Fig 31 —Srivakantham Anicut Tampraparni River Length 1 380 feet Discharge 123 000 second feet.

(62) Fig 32 is the Pelandorai Anicut, likewise a Madras work The head is 9 feet, consequently a base length or a value l of 108 feet is required

* But if this rubble up stream were scoured away as occurred at Narora and at Khanki and not replaced the work would come to grief in the same way as happened at these places See Editor s Notes at end of this Chapter —Ed

(the coefficient being 12) The actual value is about half this, proving that the work, like the previous one, is founded on clay.

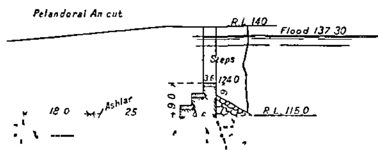


FIG 3.—Pelandorai Anicut Length 860 feet Discharge 850 000 second feet

For falls on clay or rock the length of floor or apron should $= 3H$, or, if there are shutters, $3H^a$ The riprap or pitching beyond should, it is considered, extend at least as far again

(63) We have hitherto dealt with the section of the weir itself, but canal head works comprise, in addition to the weir itself the following distinct items which may be enumerated as —

First The weir across the river
Second The weir-scouring sluices or under-sluices
Third The canal head or intake

With regard to the position of the weir relative to the river, the first point to be determined, in deciding on the position of a canal take-off from a river, is the most suitable site for the purpose

The following remarks excerpted from Mr Buckley's valuable work The Irrigation Works of India will explain the points to which attention must be directed in forming an opinion on this matter —

The selection of the site of the head works is often a matter of considerable difficulty One of the most essential points to be first considered *is the nature of the silt carried by the river whether it is fertilising or the* reverse what proportion of it is desirable to carry down the canal and whether the soil in which the canal is to be cut can stand the velocity which can carry the silt When the slope of the canal is determined an idea can be formed from the levels of the country at what point the water can be delivered on the surface of the ground and the area under command of different sites for the head works can be ascertained The approximate length and depth of cutting of the unprofitable portion of the canal which lies between the head works and the first point of irrigation can also be roughly worked out, and it is necessary to do this as the cost of this portion may often be very great The height of the weir above the bed of the river should next be determined This will depend on the depth of water required in the canal and the level of the canal bed with reference to it Then the effect produced by weirs of various heights on the flood level of the river

should be worked out, the maximum flood discharge must be ascertained from gauge readings, from cross sections of the channel and known surface slopes, and from these data the afflux, or height by which the flood level will be increased by the obstruction caused by the weir, can be calculated This point is of great importance, as upon it depends the necessity for constructing embankments to control the river above the weir, so as to prevent inundation and the possibility of the river outflanking the weir The most suitable position as a rule for a weir and head works of a canal is on a portion of the river where the channel is straight, the velocity uniform, and the sectional area of the stream fairly constant A narrow gorge of a river appears to have the advantage of cheapness, but it may prove the most expensive owing to the greater velocity and depth of the current necessitating a heavier section than usual A particularly wide reach of a river has, on the other hand, the disadvantage that the average velocity being decreased the deposit of silt and sand is encouraged, and the bed of the river above the weir is likely to become so raised that difficulty will be experienced in keeping a channel open to the take-off of the canal Another point of importance is to select a site as near as possible to a stone quarry, so as to diminish the lead of building materials "

(64) Where the river is of abnormal width, consisting of wides and banks overflowed in flood time only, the channel can be narrowed with advantage, care being taken not to reduce the length of the weir below that of the average width of the river at sites where high friable banks occur.

This reduction in width will generally take place on one side only, viz , on the opposite bank to where the canal off-take is situated This is effected, by the construction of a T head composed of earth or even sand of considerable dimensions and pitched with stone , an example of this is given in the plan of the head works of the Sidnai Canal, in which the T head is shown (*vide* Fig 23) Up stream, training banks or spurs will also be necessary as a support, as inspection of Figs 8b and 9b will show

(65) A difficulty experienced with weirs over very wide, sandy river beds is the formation of islands of sand in the bed above the weir These islands get soon overgrown with weeds, and if not removed form a serious menace to the safety of the weir by causing cross currents which, if they run parallel to the work, may undermine it The best preventive of this danger is the adoption of collapsible iron shutters fixed on the crest of the weir * The supports consist of iron rods hinged to the crest of the weir The leakage through them is insignificant The weir then need not be built as high as would have been necessary prior to the introduction of shutters, and so does not offer near so much obstruction to the water section of the river when in flood

In the older works, such as the Okhla, Narora, and Dehri Weirs, the depth of the crest shutters did not exceed 3 feet In more recent works, and also since fitted to older weirs, much deeper shutters are used In the Rupar and Chenab weirs 6 feet deep shutters are used, as also on the Betwa and Dhukwa weirs

* But see Editor s Note on Weir Shutters (PP) at end of this Chapter —ED

EDITOR'S NOTES

(M) *The Lessons of the Narora Weir*—The author performed a useful service by systemising in graphic formula a theory of design based on certain principles which had for some time previously received considerable recognition in North India The very neatness of his process however shows the difficulty of accounting for the accident to the Narora Weir in March 1898 by the hypothesis of hydrostatic uplift or blow up It may be well therefore to lay before the reader evidence officially recorded which has not hitherto been adequately published

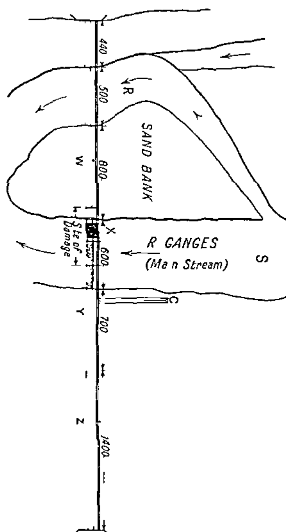


FIG 33—Narora Weir in October 1876

In the winter of 1875–76 the foundations of the Narora Weir were laid and built up to about floor level and the summer floods of 1876 were allowed to pass over the same The foundation of the crest wall consisted of a line of masonry walls sunk to R L 565 or 10 feet below floor level After the flood season it was found that where this line crossed the deep channel of the river (point marked *X* in Fig 33) the masonry had subsided owing to underscour (see Fig 34) At this point the bed of the deep stream was normally at about R L 565 but the scour of 1876 went down to R L 559 or deeper The work should have been made especially strong at a weak point like this by sinking the foundations deeper or by throwing abundance of loose stone reinforced by sheet piling into the deep channel up stream but nothing of the sort was done The only safeguard adopted was the laying of an apron of puddled clay covered by a 3 foot depth of loose blocks of *kankar* * along the whole length of the weir up stream of it Nothing special was done to strengthen the deep stream section and this neglect appears to have been mainly the cause of the accident twenty years later The clay and kankar apron was effective so long as it was kept in good repair and maintenance but even this duty was neglected Construction was completed in 1878 and in the Completion Report of 1897 the work was

* *Kankar* A soft hydraulic limestone

depicted as in Fig 35 (but, of course, without the stand-pipes, *a'*, *a"*) The dwarf-wall, shown in dotted lines, forming a cistern or water-cushion below the drop, was not built. Fig 33a shows that the river, in 1878, approached the weir obliquely from the left, a fact which implied diagonal scour towards the Weir Sluices and Canal Head at the right flank

In 1895, the engineer in charge of the weir set his heart on sluicing away

Fig 33a —The Ganges in 1878 at Narora

the large sand bank, or island, above the right-centre of the work (see Figs 33, 33a), by manipulating the shutters on the crest of the weir during the flood season He succeeded only too well, and built groynes *C*, *F* (Figs 36, 36a) projecting from the weir 600 feet up stream, by way of protecting it from parallel scour In 1895 a scour 35 feet deep was sounded within 40 feet of weir crest in the flood season (Fig 37), and, in 1897, similar scour was observed about 150 feet from crest Throughout the floods of 1895 96 97, the local engineer kept shutters erect on a length of about 800 feet on the left flank, with a view to scouring away the island on right-centre, but in place of that island a new one formed on the left flank, in front of the obstruction A current was set up on the left flank which scoured the tail of the whole weir from left to right, and to check this the engineer proposed, in 1897, to build out another groyne *D* down stream of *C* (Fig 36) He described it as ' a rush of water parallel with weir, causing it to scour deeper year by year The bed of the channel is now 4 feet below the kankar talus ' Colonel Corbett, the Chief Engineer, Irrigation Works United Provinces, expressed



Fig 34

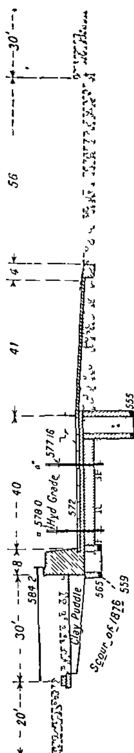


Fig 35

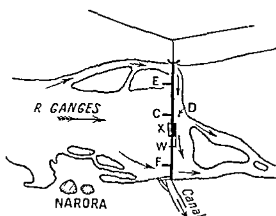


Fig 36

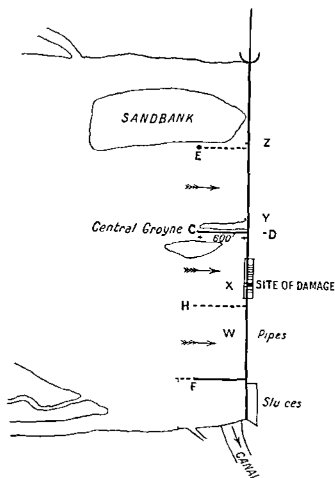


Fig 36a —Narora Weir in August 1896

some alarm at these activities of the engineers at Narora, but left his remarks ' subject to criticism by the local officers, who watch the floods and are best in a position to judge " It is curious that neither he nor the local engineers, seem to have realised that *the diversion of river currents, by manipu-*

lation of the weir shutters, might sweep away the clay and kankar apron of the weir, at the same time that it scoured away the sandbanks that rested thereon! During the low-water season, from November to March, nothing was done towards repairing or strengthening this apron, or towards ascertaining whether any repairs were necessary When the weir collapsed on March 29th, 1898, it was found that the whole of the apron in this locality had been swept away, leaving hardly a vestige of the clay-puddle underlying the kankar, and then the engineers puzzled over speculations about "blow-up" and "piping," instead of assigning a more prosaic and simple explanation to the accident

In 1896, in consequence of repeated subsidences of the Khanki weir on the river Chenab, Colonel Clibborn experimented at Roorkee on the pressures and velocities of water percolating through pipes filled with sand, and his experiments were widely discussed in India in 1897 *Apropos* of this discussion, Mr J S Beresford, officiating Inspector-General of Irrigation in India, in March, 1898, caused two vertical holes to be bored through the floor of the Narora Weir at the point marked *W* in Figs 33 36, and stand-pipes fixed in them, as shown in Fig 35, by way of ascertaining

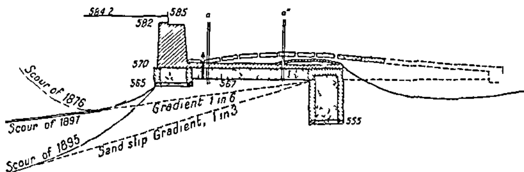


Fig 37

the hydrostatic pressures under the floor This work was completed on March 26th, 1898, when the pipe *a'* showed a pressure-level of 578 03, or 6 feet above floor surface, and 11 feet above floor bottom The floor consisted of a 4-foot thickness of concrete and brickwork, weighing 112 lbs per cubic foot when dry, or, say, 135 lbs (112 × 1 2) per cubic foot when saturated with water, and 1 foot thickness of Delhi ashlar, weighing *160 lbs* per cubic foot, dry Total weight of floor per square foot = (135 × 4) + 160 = 700 lbs The force of hydrostatic uplift at pipe *a'* would be 11 × 62 4 = 686 lbs per square foot of floor, so that the floor was sufficiently heavy to be stable through gravity alone Besides which there was its strength of resistance to shear, or to bending transversely There was nothing in these data to account for the accident to the weir at the point *X* which occurred three days later when the crest-wall of the weir subsided through a length of nearly 400 feet, and the surface ashlar of the floor had been forced up to a maximum height of 2¼ feet, tearing up with it one course of the brickwork beneath it (Fig 37a) This sketch was drawn by the Superintending Engineer, who was on the spot at the time of the accident, and the arrows indicate his conception of how the damage was caused He wrote I noticed a strong spring playing like a fountain close to the wall, throwing up sand and water, and very strong springs, like

geysers bubbling up through the grouted pitching (down stream of the deep wells) Floor had been uplifted to a maximum height of 2 23 feet and was standing arch like The only row of stones not uplifted (in fact it had sunk with the weir wall) was *that at the toe of the wall and was thus held down* by it Down stream of the floor a large hole about 8 feet deep had been scoured in the bottom of which was lying the *debris* of the grouted pitching and to left of this the grouted pitching near floor was tilted up on edge A horizontal crack was visible right through the wall at a distance of 6 feet below crest and ¼ inch wide There was no vertical crack anywhere The main cause of the disaster was undoubtedly the absence of a good puddle and kankar apron up stream of weir and insufficient length of original apron I think it most likely that the pressure of the water was divided into two portions one which went under floor and came out between the deep walls and blew up and scoured out the talus the other coming up along the

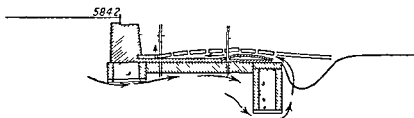


Fig 37a

smooth joint of the up stream block and then running horizontally between stone and brickwork lifted it up in part along with the stone

It seems to me that the stone floor and one layer of brickwork were uplifted by the force of the water acting in a reflex direction backward before the talus (grouted pitching) was lifted

The following difficulties lie in the way of acceptance of these surmises (*a*) If piping had occurred as sketched by his arrows the down stream line of wells should have subsided but they didn't (*b*) The grouted pitching shown as concrete 1 foot thick resting on stone rip rap in the plan of 1892 (Fig 35) can hardly have been heavy enough to offer greater resistance to hydraulic uplift than the ashlar and brick masonry of the floor

Colonel Corbett wrote in May 1898 The flooring blew up as described by the Superintending Engineer Excavation down to R L 573 at injured portion of the weir has revealed no trace of the layer of clay puddle and kunkar riprap 30 feet wide up stream of the weir To the want of an efficient apron up stream the failure of the weir must in my opinion be chiefly attributed It is impossible to be certain of what occurred but the following seems most likely The scouring out of the up stream clay apron, and the retrogression of levels amounting to from 2 to 3 feet down stream of the weir led to an increased upward hydraulic pressure on the floor and grouted pitching This latter was lifted bodily and a portion blown out through the hole thus formed a considerable volume of water found its way carrying sand A cavity thus formed under floor and crest wall the latter subsided and carried with it the adjoining portion of the floor Owing to the subsidence a horizontal crack formed in the floor between the stone and brickwork Water under a head of 12 or 14 feet found its way through the

crack, chiefly along the vertical joint between the concrete and the up stream line of walls, and lifted the stones bodily The stone floor was thus blown up carrying with it, in some places, the upper layer of brickwork There is a slight vertical crack through the concrete, parallel to, and about 25 feet from, the crest, otherwise this portion of the work has not been injured"

The difficulties in the way of Colonel Corbett's explanation are (a) If *piping had occurred under the deep wells, they would have subsided*, (b) if there was retrogression of levels down stream, *i e*, sinkage of bed and water levels, that would imply a decrease, not an increase, of hydraulic uplift against floor and pitching, (c) in any case, "piping" is the antithesis of 'blow-up', piping would have *relieved* uplift pressure, both could not occur together, (d) As regards "blow-up," if this had occurred, the whole floor, 5 feet thick, would have gone up, not the ashlar course and upper brickwork only, (e) as regards 'blow-up," if it occurred it would have been most violent at the *up stream* end of the floor where the water-pressure level was highest, whereas the ashlar was lifted highest at the down stream end of the floor where the buoyant force was weakest

Mr N F Mackenzie was one of the engineers employed on the repairs to the Narora Weir and his opinion of the affair is quoted, in Wegmann's well known treatise as follows —

The author s opinion is that the floor was first undermined by 'piping,' the concrete or perhaps concrete and masonry, settled away from the ashlar leaving a horizontal joint into which the water found its way, and this probably occurred when the up stream and down stream water was at about the same level To resist a head of 11 feet of water, when ponded up, there was only 1 foot of ashlar left, and it accordingly blew up The talus stones were also undermined and shaken, and were then ripped up by rush of water through the rent in the floor, and were not blown up by hydrostatic pressure When the floor first settled it would probably crack at the toe of the crest-wall thus accounting for the strong spring at that point The sand blowing in the talus is also accounted for by piping, as the removal of most of the foundation sand under the floor means that there was little friction to reduce the velocity due to the head The theory of piping and settlement is quite as consistent with the facts as the blow up theory, and for this reason the local engineers are by no means positive as to the cause of the accident

At the Simla Engineering Conference of 1913, Mr C H Hutton, Chief Engineer Irrigation Works, United Provinces, read a paper on this subject He mentioned that since the accident of 1898, pressure pipes had been inserted for observation in the Narora Weir, not only at site *W*, but also at *Y* and *Z* (Figs 33 36a), and not only in the floor, but also in the up stream apron, and down stream of the lower curtain Curiously enough, however, no similar action was taken at or near *X*, the site of the damage of 1876 and of 1898, and of the crossing of the former deep channel of the river Mr Hutton exhibited the results of the observations of pressure levels and inferred that they agreed generally with the "hydraulic gradient' theory, as adopted in Bligh s *construction, except that the path of percolation* seemed to follow the line of least resistance,' instead of the line of creep" In other words, according to him, in the case of a horizontal floor, held in by two lines of sheet piling, the percolation, instead of creeping down one side and up the other of the up stream piling then along the floor, and

then down and up along the down stream piling, would pass direct from the toe of the up stream piling to that of the down stream piling

He noticed, however, that the pressure pipes at W, on March 26th, 1898, when there was (apparently) no up stream apron existing, showed pressures 2·7 feet higher than those indicated by the line of hydraulic gradient, as drawn, either by Bligh's process, or otherwise It is possible that at W on that date the up stream apron, 30 feet long, may have existed, but neither Hutton nor any other commentator with knowledge of the conditions at Narora has even hinted at such a possibility There was nothing in the evidence of pressure data produced by Hutton that could support the theory that the damage to the weir in 1898 was caused by hydrostatic uplift

In Fig 38 the section of the Narora Weir is drawn to an exaggerated vertical scale to show the hydrostatic levels in the pressure pipes passing

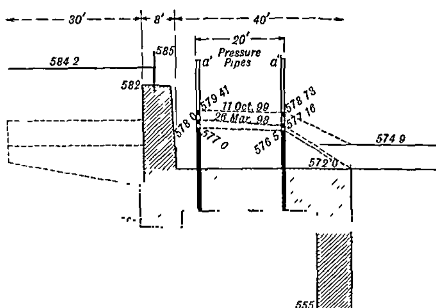


Fig 38 —Pressure pipe Records, Narora Weir

through the floor on three dates viz, on 26th March 1898, three days before the accident, and on the 11th and 23rd October, 1899, after the damage had been repaired On the first and last dates the water surface level of the river, down stream of the weir, was at about 572, but on the 11th October it was nearly 3 feet higher On the first two dates the river s water surface up stream of the weir was approximately the same viz, 584 2 or 584, but the 3-foot rise of water level, down stream, on the 11th October had the effect of raising the water levels in the pressure pipes by about half that height, viz, 1 5 feet It is noteworthy that the difference of water-level in the two pressure pipes (which were 20 feet apart) on all three dates averaged about 0 7 foot, indicating a hydraulic gradient through the subsoil of about 1 in 29

To sum up the case, we invite the reader to bear in mind that the damage to the weir in 1876, and in 1898, occurred at the same place viz, where the weir crossed the quondam deep channel of the river Well foundations are supposed to rely largely for stability on frictional resistance of soil, but the

wells of the crest-wall were not "sunk" in the soil, they were merely built up "in the air," resting on the bottom of the deep channel, at R L 565 The floor, with bottom at R L 567, must have been laid on *artificial sand-filling*, which was liable to subside, in any case The river bed, in 1876, was scoured down to at least R L 559, and probably much lower, and sand from under the floor and wall must have slipped into the scour, leaving cavities behind for which no remedy was adopted, beyond the laying of an apron of clay and kankar, 30 feet wide, up stream

In 1895 the local engineer began manipulating the weir-shutters as a means of scouring away the up stream islands, and incidentally scoured away the up stream apron Hutton has recorded that a scour 35 feet deep was measured within 40 feet of the weir crest Experience in pumping out foundations in sandy soil have shown that, below spring level, saturated sand will flow towards the sumph along gradients as flat as 1 in 3, or even flatter, and a glance at Fig 37 will probably satisfy the reader that sand from under the floor, at R L 567, would readily flow into the scour bottom at R L 535 or thereabouts On the subsidence of the floods the scoured hole would partially fill up by deposits from the falling flood, but cavities would probably remain under floor and drop-wall Hutton mentioned that, after the accident of March, 1898, a "hole 8 feet deep' (R L 559 ?) was found near the toe of the foundation of the crest wall (see vertical arrow-head in Fig 37) It is possible that this cavity may have existed ever since the accident of 1876 for both Corbett and Hutton have recorded that *probings made throughout the weir floor in* 1898 *showed no cavities except in the vicinity of the subsidence* i e, at the site of the crossing of the quondam deep channel of the river Early in 1897 the central divide groyne" at *C* was built, and during the flood season of that year the local engineer reported that It was brought into severe action during the process of erosion of the remains of the old island A strong current, parallel with weir and about 150 feet from it set up after the erosion of the island and scoured holes 30 feet deep After the flood season, *during the winter of* 1897–98, *nothing was done to repair the up stream apron, or even to ascertain whether it still existed* The weir wall and floor were very probably lying, as beams across cavities if we may judge from the horizontal crack in the former, 6 feet above floor level, after the accident, and from the horizontal shear of the ashlar course—the compression flange of the floor slab The wall and floor were supported by hydrostatic buoyancy until the end of March, 1898, when the water levels had sunk too low for adequate support The work failed *not from hydrostatic uplift, but from the very opposite cause, the withdrawal of hydrostatic support*

So, also, the grouted pitching below the down stream curtain, failed from subsidence, not from blow-up', for the local engineer had reported a considerable rush of water along the tail of the weir, which had scoured the bed to a depth of 4 feet below the kankar talus (say, R L 562) After the accident of 1898 the authorities acted in panic, and laid a new apron of clay and kankar, extending 80 feet up stream of the weir, with sheet piling to box it in throughout the length of the work It would have sufficed merely to repair, or replace, the old 30 foot apron, and to concentrate on strengthening the 400 foot length of weir at the deep channel crossing, for nowhere else had the work shown any weakness

Clay puddle was unnecessary, its adoption being based on ideas of

percolation piping and uplift The Okhla Weir across the River Jumna had shown that loose stone thrown into the river bed proved adequately staunch with [illegible] ons

In short [illegible] original design and [illegible] he weir shutters and [illegible]

The author's graphic delineation of hydraulic gradients is based on the assumption that the extension of the clay and kankar apron to a distance of 80 feet up stream of the weir with a curtain of sheet piling was precisely what the case required But the weir never exhibited any weakness so long as a 30 foot apron without sheet piling existed so that the foundation of the formula making $c = 15$ in par (9) seems questionable If c depends on the coarseness or fineness of the sand in any particular locality it is obviously desirable that samples of sand taken from the Ganges at Narora from the Chenab at Khanki from the deltas of the Madras rivers and from the Nile in Egypt should be analysed and compared *inter se* on some uniform system

(N) *The Khanki Weir* Par (35)—The weir across the River Chenab at Khanki as originally constructed consisted of a brick masonry wall 8 feet thick and 8 feet deep and about 4 000 feet long built across the river with a masonry pavement 4 feet deep sloping downwards at 1 in 15 in the down stream direction from it to a distance of 58 feet Further down stream the slope of 1 in 15 was continued for a distance of 44 feet down to elevation 715 by a layer of stone riprap 4 feet deep held in at tail by two lines of boulder filled crates and beyond that was a talus of concrete blocks succeeded by loose stone Up stream of the masonry crest wall 8 feet × 8 feet there was an apron of loose stone sloping down at 1 in 3 upstreamwards

On the left flank of the weir and in line with it were its sluices having twelve vents each 20 feet wide with floor at elevation 715 And adjoining the sluices and up stream of it on the left bank of the river was the Head Regulator of the Chenab Canal The weir was divided into eight bays each about 500 feet wide by masonry piers carrying a wire ropeway for transit of passengers across the river Bay No 1 of the weir on its left flank was built across what had been one of the main channels of the river known as the Palkhu channel and Bays Nos 5 and 6 similarly crossed what had formerly been the main western channel *The weir should have been built of special strength where it crossed these channels but just as in the case of the Narora Weir this was not done and just as in that case failures of the structure occurred at these sites* The top of the main wall of the weir was laid at elevation 722 a level which was actually about 2 feet lower than the mean bed level of the waterway of the river even after it had been contracted in width to about 4 250 feet by the weir and its sluices The work was begun in 1890 and completed early in 1892 but even during its construction the main wall in Bay No 1 cracked and subsided when it had been built up 4 feet only This damage was not duly reported but it was discovered during the repairs to the damage of January 1895

On the crest of the weir were fixed iron shutters 3 feet wide and 6 feet high hinged to the crest which could be erected to raise the water surface of the river in order to feed the canal or laid flat to pass floods without obstruction

In July 1893 occurred the second highest flood that has ever been

recorded at this place The flood formed "standing waves" on the down stream pavement of the weir, and tested it severely The up stream apron was scoured away, the crest wall was underscoured, and about 55 feet of its length in Bay No 5 collapsed It is undisputed that this failure was due to erosion by the river currents solely

In January, 1895, the crest-wall in Bay No 1, over a length of 250 feet subsided, the sinkage amounting to 6 7 feet where deepest The engineer in charge reported on January 6th "The river was held up on evening of 5th up to 726 and thirty-four shutters of Bay No 1 were dropped with a view to washing off a high silt deposit from the hole in the weir talus which had to be filled with rubble During the night the weir crest subsided, and with it the up stream reverse slope, but the down stream slope suffered in less degree"

The executive engineer surmised that the subsidence was due to "piping" under the structure, but the Chief Engineer (Sir Thomas Higham) did not accept this view, on the ground that "piping" needed an outlet, which was not visible in this case and also on the ground that in case of piping the damage down stream would have been greater than that up stream The Chief Engineer recognised that the weir had been underscoured by the surface floods of the river and also that the damage might have been due to the rush of water through the gap where thirty-four shutters had been dropped, but he came to no final conclusion on the subject

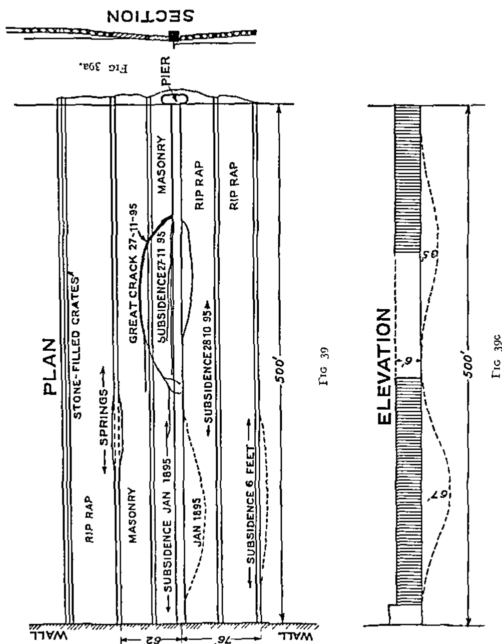
Whilst this damage was being repaired, a flood, occurring on February 2nd swept away the protective embankments, and damaged the works again

In the course of repairing the damage the loose stone apron, sloped at 1 in 3 up stream of the weir, was replaced by another 4 feet thick, extending 72 feet up stream and protected by sheet piling 17 feet deep, down to elevation 701

On October 26th, 1895, the crest wall in Bay No 1 again subsided over a length of 124 feet, just to the right of the preceding failure During the flood season, August 1895 some of the piles of the up stream apron were seen to float up and away, indicating erosion to a depth of 21 feet below crest of weir Soundings 70 feet up stream of the crest showed that the end wall had sunk 10 feet in two places *The Chief Engineer remarked that the set of the current had been very violent here, removing all loose stone up stream of the sheet piling*

When the subsidence occurred in October, cracks appeared in the masonry pavement down stream of crest, and springs bubbled up below the wall 60 feet down stream of crest, where also some subsidence took place Sir Thomas Higham remarked "There is no doubt that, whatever the cause, the weir was then undermined in Bay No 1" By way of preparation for repairs the shutters were erected on the crest, and an earthen embankment was thrown up *down stream* of the damaged bay, so as to form a still-water pool over the damaged masonry This *bund* was completed on November 28th and ponded up water to elevation 720 6 down stream of the crest, up stream of which the water level was 721 0 Under these conditions a more serious subsidence of the weir crest (maximum 3 2 feet) occurred the same day, over a length of 164 feet, the masonry pavement down stream to a distance of 40 feet, also cracking badly The plan and sections of the damage occurring in 1895 are shown in Figs 39a, b, c

Fig. 39b shows, to an exaggerated vertical scale, the conditions prevailing in 1895. The water-level up stream in October was 724·9, and down stream 720·2. Head of pressure 4·7 feet Under this head springs bubbled up



below the wall 60 feet from the shutter-line to a height of 721, as shown by an arrow-head The path of percolation-flow, by Bligh's process, would be 84 feet, and the hydraulic gradient 1 in 18, which, according to him, would imply stability Evidently, therefore, it was not a case of "piping" due to excess of gradient through sand There must have been cavities, open

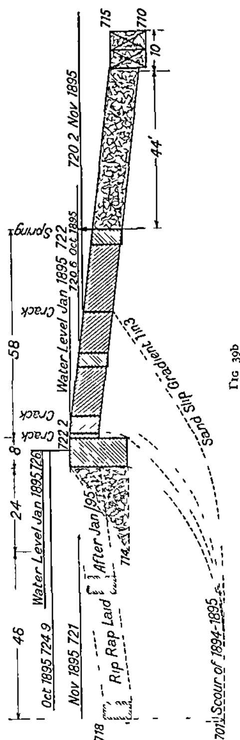


Fig 39b

under the masonry, due to the foundation sand slipping into the deep scours up stream of the weir

The reader will bear in mind that these accidents to the Khanki Weir occurred only at the weak points, where it crossed former deep channels of the river, in Bays Nos 1 and 5 The weir was a simple masonry wall, laid across the river, with bottom at R L 714, or 1 foot lower than what was then supposed to be the bed level of the deep stream Actually, afterwards, the bed was scoured to a depth of R L 701, or lower, since the sheet-piles were uprooted from that depth Down stream of the crest the structure was fairly well protected by a glacis of masonry 4 feet thick and 58 feet long, with another 54 feet length of stone riprap, 4 feet thick, beyond that, but up stream of the crest there was, in January, 1895, *only a talus of loose stone* extending 24 feet or so The scour would thus be 714 — 701 = 13 feet below foundation of weir crest, and the subsoil (which, under the pavement, at least, must have been artificial sand filling at from R L 718 to 714) might easily slip down into the hole at a slope of 1 in 3 That this did, in fact, occur is apparent from the records The subsidence was greater in the up stream than in the down stream direction Higham wrote, as regards the theory of "piping" '*Ex hypothesi* the wall could not have sunk till sand below had found an outlet, and scour and settlement must have occurred at the down stream not the up stream end But there is no trace of any such outlet The dropping of 34 shutters, with a head of 4 feet, no doubt caused a rush towards the weir, but it is hardly possible to suppose that this would have been sufficient to underscour the crest in so short a time The cause of this subsidence must, therefore, remain an open question till more light is thrown upon it " We suggest that the explanation is that the " outlet "

FIG 40—The Jhelum Weir at Rasul

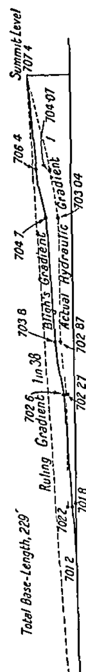


FIG 40A

to the subsoil sand of the weir lay in the deep scour up stream of it. We have remarked that the hydraulic gradient downstreamwards could not,

either in January or in October, 1895, account for "piping" towards an outlet in that direction The further damage, which occurred in November, 1895, could not possibly be attributed to piping, since the difference of water level (721 − 720 6 = 0 4 foot) up stream and down stream of the weir was insignificant Nor could it be imputed to "blow-up" We submit to the reader, therefore, that the damage in all the cases cited above was *due solely to up stream erosion undermining a work and establishing cavities under it in places where the masonry was founded upon artificially filled sand* Faulty manipulation of the shutters, practised with a view to scouring away sandbanks, must also bear a share of the blame for the accidents

(O) *The Jhelum Weir at Rasul* (par (39))

Fig 40 shows a section through the weir across the River Jhelum at Rasul It is similar to Fig 9, but shows the hydraulic conditions under a water-level of only 707 4 up stream The vertical scale has been exaggerated to double the horizontal scale for the sake of clearness In January, 1916, we bored holes through the pavement of the weir at five places along a line at right angles to the crest-wall and found the subsoil water pressure levels to be as shown in the diagram

The hydraulic gradients thus ascertained are compared in Fig 40a with those given by our author's process We infer that there may be an initial loss of head in entry into the soil (corresponding to that which we know occurs at the entrance to pipes) which our author's process does not take account of

In the case of the Narora Weir, just before the accident of March, 1898, the pressure pipe *a'* (see Figs 35, 37) show a loss of 6 feet of head, when according to our author's process it should have been only 1 5 feet in the absence of a 30 foot apron up stream, or 3 feet if such apron had existed One would expect the "loss of head in entry" to the soil to vary with some function of the head of pressure

(PP) *Weir Shutters* (par (65)) —These are a device dating from the "dull dark days of irrigation canal science in India The engineers had no reliable information as to the maximum flood discharges of the rivers across which they had to build their weirs, so they wisely designed their masonry walls too low, by way of error in the direction of safety, and supplemented them with iron shutters, hinged to crest, capable of holding up the lower water levels of the river when erect, and of lying flat on crest so as tc [illegible] trouble-some tc [illegible] ge them during [illegible] Chenab at Khanki, or of the Jhelum at Rasul, there were about 1 300 shutters, each 3 feet wide and 6 feet high, requiring a force of thirty or forty men to manipulate them The weir shutters have been partially or wholly responsible, for most of the damages that have occurred to river weirs in North India They have continued in use, although fifty years ago the Superintending Engineer, Godavari Delta Canals, wrote as follows. "I have disallowed the so-called self-acting shutters These I consider utterly useless on our canals, carrying so much silt in the irrigating season It is certain that those we have are worse than useless, in theory nothing can be more perfect, in practice, nothing less so"

In the year 1918 the shutters on the Jhelum Weir at Rasul were dismantled and the masonry weir built up 4 feet higher, without shutters

CHAPTER VII

WEIR SLUICES

(1) In all weirs constructed across wide rivers having sandy beds, or much silt in suspension, the provision of weir sluices is a necessity.

The function of these works is two-fold. First, to train the main stream of the river, the natural course of which is modified by the weir, past the canal head, which is located adjacent to the weir sluices, and to retain it in this position; otherwise in a wide river the low water channel might take a course distant from the canal head, leaving the latter shoaled up. Secondly, to keep the river bed in front of the canal head scoured down to the level of the floor of the sluices. For this purpose it is necessary that the weir should obstruct the flow of the river sufficiently to maintain an afflux that will ensure the necessary scour throughout the season when the current is heavily silt laden.

(2) The cill or floor of the weir sluices is laid generally at the deepest bed level of the river, or as near that level as convenient.

(3) The ventage provided has, in the past, in important Indian works, been usually fixed arbitrarily, without calculation of functional discharge capacity. It need not be wider than may be sufficient to pass twice the full capacity of off-taking canals under a moderate afflux, say 2 feet. In one case, that of the Laguna Weir, Fig. 13a (*post*), where the river low supply is deficient, the weir sluices are designed to take the whole ordinary discharge of the river excepting high floods. This is with the object of maintaining a wide, deep channel which may be drawn upon as a reservoir. In such a case the weir sluices form a " barrage " of the Nile pattern across the normal flow of the river, whilst the weir itself becomes merely a spillway for surplus floods.

(4) The efficiency of weir sluices in the prevention of silt deposit in the neighbouring canal head might be said to be ineffective without the co-operation of the latter work. Of late years it has become recognised that the cill of the canal head, or intake, must be located several feet higher than that of the weir sluices, and that it must be supplemented with a movable cill, formed of gates sliding in vertical grooves behind it and capable of rising or falling *pari passu* with the water surface levels of the river, so as to skim off always " top water " containing as little heavy silt as possible.

Another common feature is the provision of a ' divide groyne ', that is, a stone-faced partition embankment running out up stream, as a continuation of that abutment of the weir sluices which adjoins the weir itself. This feature was originally designed as part of the system of silt-control practised

at the Rupar Head Works of the Sirhind Canal Where that system is not practised the only function of the divide groyne is to discourage currents from flowing parallel to the weir, and close to it, towards the sluices; as such currents are liable to undermine the weir For this purpose of weir protection the divide groyne need not be more than, say, 200 feet long

(5) As the object of a weir sluice is to pass water at a high velocity, in order to scour out deposit for some distance up stream of the work, it is evident that the openings should be wide, with as few obstructions as possible in the way of piers, and should be open at the surface, the arches and platform being built clear of the flood level Further, in order to take full advantage of the scouring power of the current, which is at a maximum at the sluice itself, diminishing in velocity with the distance up stream of the work, it is advisable not only to place the canal head as close as possible to the weir sluices, but to recess the head as little as practicable behind the face line of the abutment of the end sluice vent

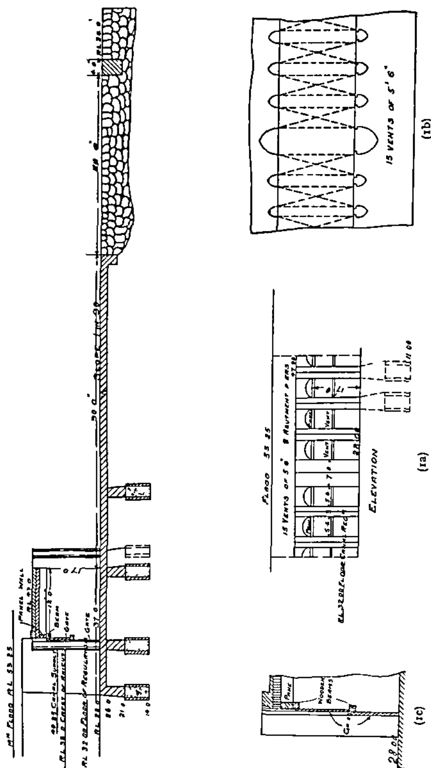
In the examples of old Indian works, which will now be given, we shall find that these conditions have often been violated, with great detriment to efficiency This in part was unavoidable, as, prior to the introduction of iron drop gates fitted with anti friction rollers the openings of weir sluices and regulators generally had perforce to be designed of narrow width, in order to admit of the gates being worked under a pressure of water Thus, prior to the year 1875, the width of waterway in each opening was limited to 5, or 6 or 7 feet only, but experience showed the necessity of increasing width to 20 or 25 feet Many old works have been remodelled on this principle, and within recent years the invention of "Stoney" live-roller gearing to gates has shown spans of from 40 to 80 feet to be economically advantageous

(6) With regard to water pressure, weir sluices perform a *rôle* identical with that of river regulators or, as they are also termed, open dams or barrages The gates are closed partially, or wholly, as a rule only in order to dam up the water to a certain level, sufficient to force supply down the adjacent canal head In the Rupar system of silt-control the sluice-gates are closed completely in order to create a still pond wherein the current may drop its heavier silt before it enters the canal When the silt so deposited has risen to a level within 5 or 6 feet of that of canal head cill, the canal has to be closed, and the weir sluices opened, in order to scour away the accumulated sand

When the river rises above canal full supply level, the gates can be lifted clear of the flood to afford a free passage to the current, but the Rupar system does not allow this

Canal head regulators or intakes on the other hand, have occasion to be entirely closed during the highest floods in the river, consequently the regulation they have to perform is entire, not partial, so that these works may be subjected to a much greater statical stress than weir sluices

Some examples of works with critical and emendatory remarks will now



FIGS 1, 1a, 1b, 1c —Dauleshwram Weir Sluices, Godaveri Canals

be given, as by this method the principles governing design can be clearly set forth.

(7) Fig 1 is a representation of the Dauleshwiram weir sluices * of the weir of that name on the Godaveri River in the Madras Presidency

This old work consists of fifteen vents of 5½ feet width, which are arched over above canal supply level, presenting a level platform 19 feet high above cill The openings are completely closed by wooden draw gates in grooves, reaching above the spring line, while the arch segments are closed by narrow panel walls which are supported on wooden beams The upper gates rest on another line of wooden cills, against which the top side of the lower gates bears The maximum flood line is 25¼ feet above the sluice cills, and so the whole work is submerged during high floods, a depth of 6¼ feet passing unobstructed over the platform

The R L of the cill is - - - -	28 00
,, , of the cill of head regulator - -	32 00
,, , of the crest of the anicut - -	38 00
Full supply level in canal - - - -	40 25
Level of submerged platform - - -	47 00
Level of maximum flood - - - -	53 25

On the occasion of floods the gates are either suspended at the top of the grooves in which position they will still somewhat obstruct the waterway through the sluice openings, or else are taken out bodily and removed

(8) The next plan to be examined is that of Fig 3, of the Sangam Weir sluices on the Penner River, Madras Presidency This is a much more modern work than that at Dauleshwiram, having been constructed in 1870–80

As will be seen by reference to the plan, the design consists of a series of small vents 6 feet wide and 5 feet high, which are topped by a heavy breast wall carried up to above flood level, which barrier effectually prevents any passage of water except through the sluice vents

The design, if remodelled nowadays would be somewhat similar in appearance to Fig 10, or 5 or to the Assiût Regulator, or Rupar Weir sluices, viz a simple bridge of a few large spans of 15 feet or 20 feet or more, with arches and platform above M F L

A noticeable peculiarity in Fig 3 is the roadway provided behind the breast wall, which is carried by a separate set of arches springing from every alternate pier, which pier is lengthened to receive them This roadway slopes down to the anicut crest, and is probably used for cart traffic

The defences of the floor against erosion or upward statical pressure are very considerable There are triple lines of interlacing curtain wells, one at either end of the masonry apron and the third at the termination of a length of very deep rubble pitching One minor point deserves mention, and that is the disposition of the dwarf partition wall dividing the weir sluice floor from that of the weir itself This wall is given an outward trend,

* These were dismantled in the year 1910 and replaced by new sluices having 10 vents each of 20 feet built on more modern lines —ED

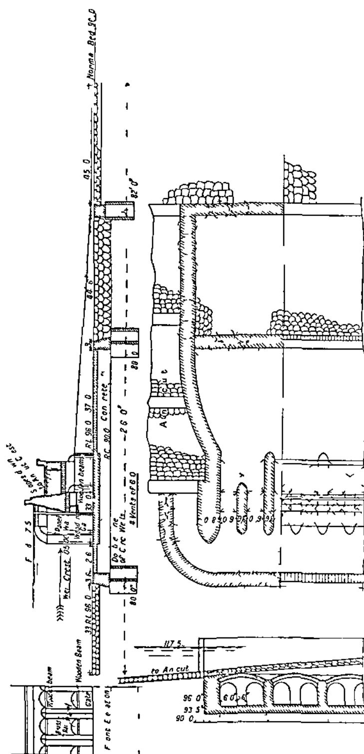
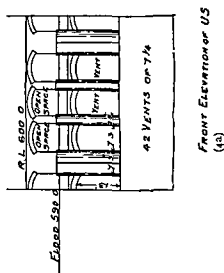


FIG 3 —Sangam Weir Sluice Pennér River

so it encroaches on the weir apron and talus This cannot be considered a good arrangement, the partition wall should be perfectly straight and normal to the weir crest If widening of the exit waterway is deemed re-

quisite this should be effected on the opposite side by throwing the flank wall inwards

(9) The maximum horizontal stress on the superstructure of a partial regulator may possibly be in excess of what is produced by the maximum statical head which means that a greater effect can be produced by a less head This is owing to the influence of the increased depth of the water fore and aft of the work We have already seen in Chaps II and VI that



Long

the same applies to weirs particularly when subject to flotation

(10) The Narora Weir sluices are shown in Fig 4 These were built about 1875–78 and present a marked advance on the last example The vents are 7 25 feet wide in groups of three spans separated by piers 7 25 feet thick The intermediate piers are 2 75 feet thick or 37S nearly There are two tiers of arches through both of which an open slit is provided for the passage of the draw gates The tra velling winch runs on top the plat

Long⁴ Section of Undersluices

Figs 4 4a —Narora Weir Sluices

form, which is 28 feet above floor level, a height sufficient to allow the gates, when raised up, to hang clear of the flood line The lower tier of vaulting forms a lower platform, 3 feet below high flood level, and the space between the piers, which continues up to the upper tier, is open, allowing the passage of water above the lower platform The upper parts of the piers are pierced by wide openings at right angles (*vide* Fig 4a) thus forming a vaulted passage, which is of great convenience for the manipulation and stacking of the gates when drawn up

It is quite evident that this design could be greatly improved by simplification The spans should be widened, say to 27.25 feet, by knocking out the intermediate piers and leaving only the abutment piers

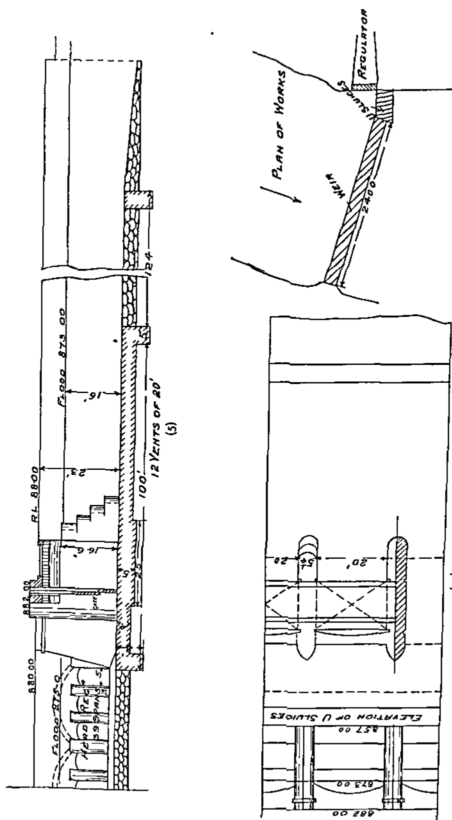
(11) The maximum statical head on the floor is 13 feet the coefficient c being 15 l will equal $c \times H$, or 195 feet (*vide* Chap VI) Up to and inclusive of the curtain wall it is 180 feet and if the grouted rubble beyond be esteemed as an impervious continuation the total value of l will be increased to 280 feet The floor thickness of 5 feet as was also the case with the weir apron (par (28), Chap VI) is deficient * It should be by formula (3) par (25) Chap VI $= \frac{4}{3}\,\frac{H-h}{\rho - 1}$ Here h, or the loss of head due to percolation is $\frac{53}{15} = 3.5$ feet $(H - h)$ then will be $(13 - 3.5) = 9.5$ feet The correct value of t will then be $\frac{4}{3} \times \frac{9.5}{1\frac{1}{4}} = 10$ feet thus the floor is in a high state of tension being only half * as thick as it should be The value of L is 220 feet that is 50 feet longer than that of the weir The correct proportion will be determined later this proving a valuable guide (*vide* par (13) (*post*))

(12) The weir sluices of the Rupar Weir (Fig 5) on the Sutlej River are at the head of the Sirhind Canal No doubt this design was the precursor of the Assiut Regulators which are built on closely similar lines The vents are 20 feet wide bridge openings the arches springing at maximum flood level The piers are $5\frac{1}{2}$ feet thick the floor of the bridge is a solid mass, being founded on clay or clay and boulders the thickness can thus be reduced below what would be requisite on a sand foundation

The superstructure consists of a platform carried by a tier of arches springing at flood level and divided as usual into two parts by the slit for the gates and grooves

The plans in Fig 5 show the work as it stood originally The crest of the weir has since been raised from 865 to 866.5 and provided with 6 feet deep collapsible shutters capable of raising supply level from 866.5 to 872.5

* Yet it has shown no weakness during the past fifty years We understand that the foundation soil of these sluices is not pure sand It seems a pity that stand pipes have not been inserted vertically in this floor by way of revealing the hydrostatic pressure in the soil under the floor —Ed



FIGS 5, 5a, 5b.—Rupar Weir Sluices, Sirhind Canal, Sutlej River.

The canal head cill has also been raised from 859 or 2 feet above weir sluice floor to 866 or 9 feet above it, and all the small jack arches and piers shown in elevation have been cleared out leaving 21 foot openings The weir sluice in elevation is shown in Fig 5a The present weir sluice gates now close to above the arch spring line so that the sluice head can be entirely closed if necessary The work is on a boulder bed with coefficient about 9 Fig 5c is from a photograph of the work

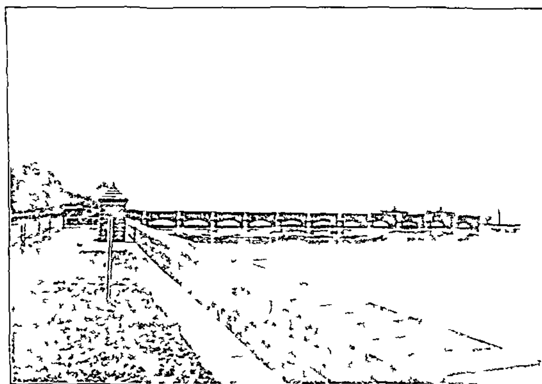


FIG 5c —Weir Sluices of the Rupar Weir at the Head of the Sirhind Canal

(13) A section and part elevation of the Chenab Weir Sluices at Khanki are given in Fig 6 These are very similar to the Rupar Weir sluices The depth upheld by the gates is 1 foot less being 13 feet against 14 feet at Rupar Here three roller gates are employed raised by a travelling winch The outside arch which has so good an appearance is here replaced by iron or steel girders which carry the outer rail of the traveller This work being on a river bed whose sand * is of class 2 with coefficient 15 we are able to compare it with the Narora Weir sluices the unit flood discharge of the Chenab River over the weir is however double that of Narora This latter item influences the value of L the width of the talus as has previously been shown in par (27) Chap VI Whereas the width of the floor apron is a function of the head both are influenced by the coefficient representing the

* We understand that neither the Khanki nor the Narora Weir Sluices have a sand foundation —ED

nature of the bed , consequently the formulas applicable to weirs will only require multiplying by some enlarging factor to render them suitable for the weir sluices

If in formula (4), par (27) Chap VI , the multiplier of c be increased from 10 to 15, the results will be as in the following comparative table

$$(L), \text{or width of talus for weir sluices} = 15c\sqrt{\frac{H^b}{10}} \times \sqrt{\frac{q}{75}} \qquad (1)$$

The following * is a comparison between the values of L of Narora and Khanki worked out by this formula with the actual dimensions —

Name	C	H^b	q	Equation	Amount	Actual Value of L.
Narora	15	10	75	$225 \times 1 \times 1$	225	216
Khanki	15	7	150	$225 \times \sqrt{7} \times \sqrt{2\ 0}$	265	300

With regard to (W) the length of the apron or floor beyond the head work, in both Narora and Khanki the length is given the same This would tally with formula (1), Chap VI , if $4c$ be increased to $7c$, the formula will then become

$$W = 7c\sqrt{\frac{H^a}{13}} \qquad (2)$$

This for Narora and Khanki works out to 105, H^a being 13 in either case

(14) The section in Fig 6 will now be examined as regards hydrostatic pressure

First as regards length of percolation Here the value of l should be $15 \times 13 = 195$ the same as at Narora , it actually measures 227 feet The thickness of the floor at the critical point just beyond the head work is only 4 feet , according to formula (3) par (25) Chap VI , t works out to 7 5 feet The floor is, therefore, in high tension † This could be avoided by making it thicker at the root, tapering to 4 feet at the end, which is the scientific profile for all floors under diminishing hydrostatic pressure Further, a rear extension of the floor up stream of the head work in the deep channel would be a great improvement and should invariably be adopted in all weir sluices founded on sand As we shall see, this precautionary measure has been carried out in the case of the Jhelum Weir sluices (Fig 7), where however, it is not so much required

One peculiarity of this work is the slope given to the apron which drops 1 in 20 down to R L 710, which is 5 feet below the level of the weir talus as also that of the cill of the bridge floor This would appear to be a good arrangement A long flank division wall separates as usual the weir sluice

* The discharge per foot run of waterway through sluices is very different from that per
louble The masonry crest of Khanki

and like Narora Sluices the Khanki

way from the weir itself This also extends up stream to beyond the canal intake, forming the ' divide wall mentioned as a desirable adjunct in par (4) (*vide* Fig 8b, Chap VI)

The concrete blocks forming the talus are a common feature in weir sluices of Punjab canals They can easily be laid in water They are 4 feet square and 2 thick, and consequently weigh 2 tons each The concrete is not cement concrete Portland cement being used very little in India on account of its cost besides excellent hydraulic limestone is obtainable everywhere in Northern India in a nodular form close to the surface where it is quarried just as gravel would be

(15) The Jhelum Canal Weir sluices at Rasūl are illustrated in Fig 7 * The permanent weir crest is at R L 707 5 The value of H^b is consequently 6 5 feet According to formula (1), par (13) the value of L works out as follows 225 $\times \sqrt{65} \times \sqrt{2}$ = 256 feet

The actual value is 255 feet so that the theoretical value agrees closely with the actual W the width of the masonry floor according to formula (2), par $(13) = 7c\sqrt{\frac{H_a}{13}} = 105 \times \sqrt{\frac{11\ 5}{13}} =$ 99 feet it is actually 108 feet Owing to the provision of a rear apron the hydrostatic pressure at the critical point is reduced from 11 5 to 3 5 feet for which the 5 foot thickness is more than sufficient

This weir is provided with a long divide wall up stream shown on the site plan Fig 9b Chap VI This wall runs parallel to the axis line of the river and to the left flank retaining wall

The down stream outer flank wall is normal to the common axis of the weir and of the weir sluices which are on one line

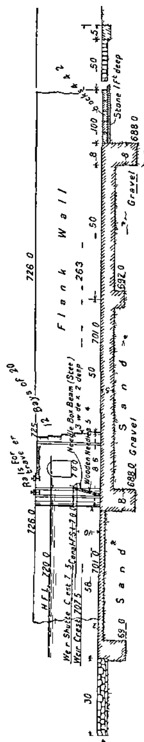
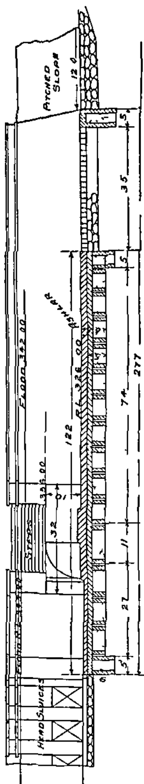


Fig 7 —Jhelum Canal Weir Sluices at Rasūl

(16) We now come to a different type of weir sluice from any of the foregoing which has only been adopted in the Province of Behar The governing principle of this design is to afford a

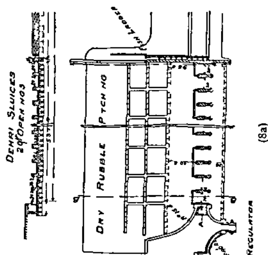
* Fig 7 is out of date The needles were scrapped twenty years ago and replaced by gates in three tiers sliding in vertical grooves and these again were superseded in the year 1917 by Stoney gates 14 feet high one to each vent of the sluices —ED



Figs 8 8a —Dehri Weir Sluices Son River Bengal

free passage of flood water between and over piers, which are only of the necessary height for regulation purposes and have no arched superstructure whatever The water is held up by collapsible gates of large size, working between the piers

The designers of these open sluice ways in the year 1875 were evidently of opinion that it was unnecessary to have the weir sluice floors at a much if any, lower level than that of the canal head regulator This has proved to have been a mistake It is absolutely necessary that the difference of floor level of these two adjacent works should be considerable



With the heavy drift sand found in these rivers a cill several feet higher would exclude such sand

(17) Fig 8 represents the Dehri Weir sluices on the Son River This work consists of twenty openings 20½ feet wide divided by piers 5 feet thick and 33 feet long These piers are only 10 feet high The openings are closed by double collapsible wooden gates 20 feet long and 10 feet high The second gate is for temporary use, and is first raised by hand which enables the real self falling gate to be hauled upright by tackle, when the temporary first gate is lowered out of the way

(18) The weir was provided with sixteen central sluices of 20½ feet width The object

in view was to prevent cross-currents and to train another channel straight on to the weir These expensive works, however, have since been built up

The block plan (Fig 8a) shows the position of the canal head, which is recessed much too far behind the weir sluice abutment The alignment of the dwarf wall dividing the weir sluice and the weir is shown on the block plan

As regards the remodelling of the adjacent canal head, see next chapter

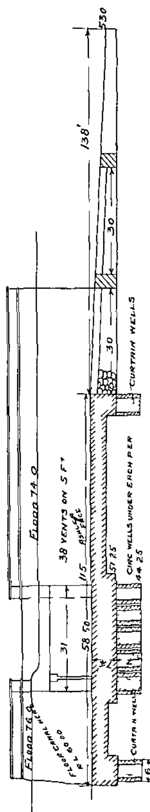
(20) The Assiût and Zifta works, to which reference has more than once been already made, are examples of partial regulators

The plans of the former are given in Fig 10

The Assiût Regulator was built across the Nile to regulate the supply in the Ibramiya Canal

This work will never be entirely closed, and the outside difference of level to be provided against is that between R L 48 40 and 45 85, viz , 2 55 metres (*vide* Fig 10) This is the head, on the extent of which the design of the floor and also the superstructure depends The value of l will then be (taking c as 18) $18 \times 2\ 55 = 45\ 90$ metres It measures actually, 40 metres horizontally together with 10 vertically, total 50 metres If the low supply were all held up, the head would be 4 7 metres, requiring 85 metres as length of percolation

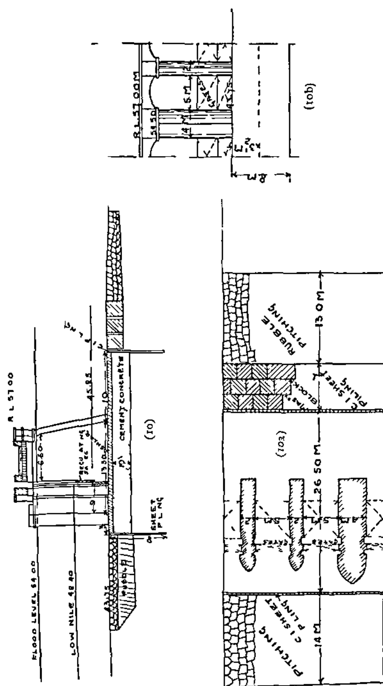
(21) With regard to the superstructure, the resultant line of maximum pressure should pass through the outer boundary of the middle third of the base of the piers This test is best arrived at by graphical process and will be worked out for the Ibramiya Regulator, and so need not be repeated here This determines the requisite base length of the piers



SECTION THRO UNDERSLUICES BELOW CANAL HEAD 38 VENTS OF 5 PIERS 4 THICK

FIG 9 —Jobra Weir Sluice Mahanuddee River Bengal

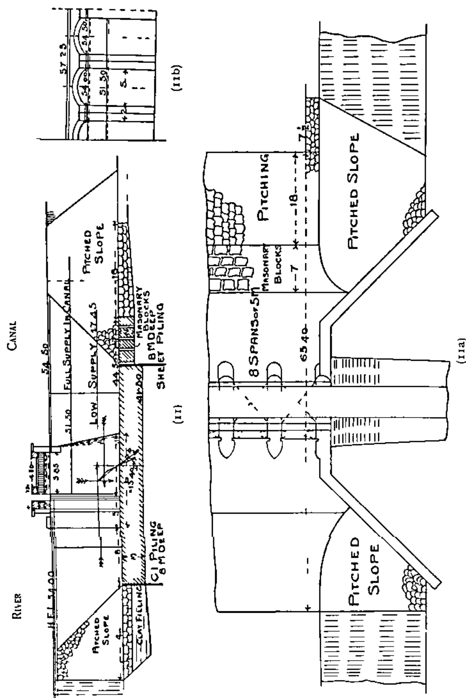
The spans are 5 metres *i e*, 16 feet, and the piers are 2 metres thick a too high proportion of 4*S*, every third pier is an abutment pier The



Figs 10 10a 10b —Assiût Regulator Nile River

general arrangement closely resembles that of the Rupar Weir sluice head but is even more simple The working platform is carried on arches springing at high flood level These arches are divided at the grooves, the open space being wider than usual A travelling winch straddles this

opening The draw gates are of steel, fitted with anti-friction rollers, stanched by loose stanching rods as employed in Stoney's patent gates



Figs 11 11a 11b —Ibramiya Canal Head, Egypt

For plans of gates grooves and travelling winch *vide* Min Pro Inst C E Vol CLVIII

The foundations were originally designed as rows of sunk blocks under

each pier, connected by a concrete floor Owing to the successful introduction of cast iron tongued and grooved piling, the use of blocks was abandoned as being slow and expensive, and rows of sheet piling each side of the floor 25 feet deep were substituted, the depth of the floor being increased to a uniform thickness of 10 feet This depth is necessary to properly distribute the weight of the superstructure to reduce the pressure to 1 ton per square foot

(22) The detached masonry blocks shown in Figs 10 and 10a are a form of construction originated in the Punjab These blocks are made of rubble masonry in cement mortar, and are laid in position when set, on the wet sand or in water by a heavy travelling crane When *in situ* the interstices are filled up with cement concrete it thus forms a solid mass of heavy masonry This arrangement is admirably suited for a continuation of the masonry floor of a regulator The advantage of the system lies in the fact that the blocks can be deposited in water, the interstices likewise being filled up by depositing cement concrete in skips, or else the work can be partially pumped dry to facilitate this operation In the Punjab the blocks are usually made with stone ballast in ordinary lime mortar, and the spaces between blocks are left open, in order to permit free settlement

(23) The plans of the Ibramiya Canal Head are given in Fig 11

This work, although a canal head, is but a partial regulator, and is consequently in a similar case to a river regulator or a weir sluice

The general arrangement of this work is identical with that of the Assiût Barrage

The Ibramiya Canal Head is, however, subject to a greater head of water than the latter work

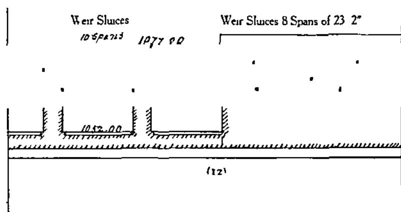
The maximum head of water to which the floor is subjected is the difference between low Nile level and the canal bed *i e*, 47 75 — 44 50=3 25 metres, so that this floor is under greater statical pressure than the Nile regulator The superstructure is also subjected to a much greater pressure than the Nile regulator, owing to the great depth of water on both sides of the gates, which pressure is a maximum in Nile flood with full supply in the canal

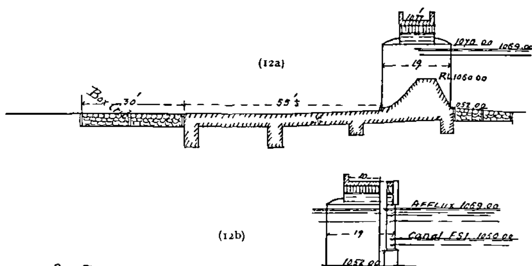
The lengths of the piers in either of the regulators are much the same, the base being 13 50 metres in each case, the top width is somewhat less in the Ibramiya work The spans and thickness of the piers are the same in both cases, but the Ibramiya Canal Head has no abutment piers The masonry floor is made wider in the canal head, being 31 40 metres against 26 50, a difference of 5 metres This is owing to the greater head to which it is subjected These two are models of good design

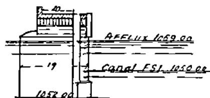
(24) The value of l in this case should be $3\ 25 \times 18 = 58$ metres The horizontal component is 45 metres, add the vertical of 10 metres, the sum is 55 metres If the floor thicknesses at each end be added in it would increase the amount to 61 metres, so that in this instance the actual closely cor-

responds with the calculated, forming a corroboration of the correctness of the value assigned to c, viz 18 In these calculations the lower line of sheet piling has not been included

(25) The previous examples are all of works founded on sand At the heads of rivers, before the stream has left the hilly, rocky country near its







Figs 12 12a 12b —Western Jumna Canal Weir and Weir Sluices

source and debouches into the plain the river bed is generally composed of rock or boulders the latter being more or less mixed with sand or shingle It will be as well to give an example of a head work thus circumstanced

Figs 12 12a 12b are reproductions of the record plans of the Jumna Weir sluices at Tajawala at the head of the East and West Jumna Canals

Of these Fig 12 is a longitudinal section through part of the work

The eight spans of 23 feet 2 inches at the east end of the west sluices have

been closed with a masonry wall up to weir crest level in order to reduce waterway and to force the current towards the canal head at the west end The afflux level is 17 feet above the floor of the weir sluices while full supply level in the canal corresponds to the crest of the weir, and is 8 feet above same level The actual statical head on the weir and weir sluice floor is 10 feet

(26) In this case, however, the boulder bed is capable of great resistance to erosion, consequently the talus can be curtailed, at the same time, owing to the great abundance of building material available at the actual site, the cost of a boulder masonry floor will be comparatively so inexpensive that the masonry apron can be carried wider than would otherwise be advisable

(27) A work built on boulder formation cannot be considered free from hydrostatic pressure below the floor The coefficient suitable for boulder beds varies from 5 to 9 In this case the actual value of l is about 90 feet, this would make the coefficient 9, which has been adopted (par (9) Chap VI) The floor is too thin for the hydrostatic pressure * The value of $(H - h)$ at its commencement is 6 5 feet, to meet this, the floor, half of which lies below the L W L, has a mean specific gravity of 1 75, and is $3\frac{1}{2}$ feet thick, t_p is then 6 1, but it should be $\frac{4}{3} \times \frac{6\ 5}{1\ 75} = 5$ feet thick

The canal head or intake connected with the weir sluices is illustrated in the next chapter According to formula (1), Chap VI, the least length of apron of the weir should be $4c\sqrt{\frac{H^a}{13}}$. H^a in this case being 8 feet, W will equal 25 feet With regard to the weir sluices, by formula (2), par (13) of this chapter, W should have a minimum value of $7c\sqrt{\frac{10}{13}} = 7 \times 9 \times\ 88 = 55$ feet, which it actually measures

(28) *In the United States a large number of canals have been constructed* in recent years, the head works of which display considerable ingenuity in general design, some are diversion weirs, as dealt with in the last chapter, of low height, but the majority are constructed over the rocky beds of mountain torrents of considerable height, intended to form storage reservoirs similar in lines to the Betwa, Periyar, and Assuan works

One example of low weir across a wide sandy river has very similar conditions to Indian works This is the Yuma irrigation project about to be undertaken, a description of which is given in ' Irrigation Engineering," an excellent American work, from which the plans have been obtained

Fig 13 is a cross section of the Colorado River From this it will be seen that the general level of the river bed is, roughly, about R L 143 0, the deepest part of the channel being about R L 131 00 The weir crest

* Yet it has shown no weakness during the past fifty years —ED

has been fixed at 10 feet above L W L *i e* at 151 00 The design of the weir is a close copy of the Okhla (Indian) type the section has been given in Fig 29 Chap VI The level of the horizontal talus where the slope ends is R L 138 00 *i e* 3 feet below L W The sluice way floors on each side of the weir are at the same level *i e* 13 feet below crest The weir is 4 500 feet long and the estimated maximum flood of the river is 198 000 cusecs (second feet)

The arrangement of the rest of head works viz the weir sluices and canal head is so remarkable as to be deserving of considerable attention The weir ends at both flanks on the existing rocky banks of the river

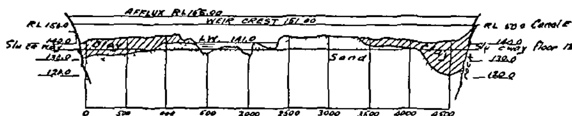


Fig 13—Section of Colorado River

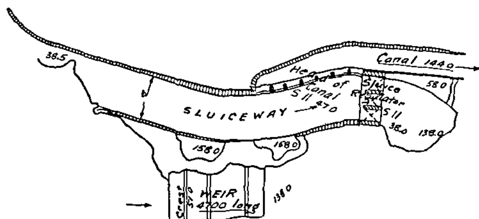


Fig 13a Laguna Weir and Left Head Works

Beyond these and separated from the weir sluice way channels are cut in the solid rock on either flank These being quite independent of the weir it is evidently a convenience to fix the position of the weir sluice head *below* the weir so that the junction of the scouring channel with the river bed will take place well clear of the latter work

Another peculiar arrangement is the alignment of the two canals which instead of being as is almost invariably the case in Indian canals parallel to the axis of the weir until the river banks are well cleared are aligned parallel to the river and at right angles to the weir axis This arrangement is necessitated by the level of the rocky ground in the vicinity of the head works The canal head vents therefore which are situated just above the weir sluices in the side of the sluice way channel discharge through the right flank of the canal *i e* at right angles to its direction

The sluice way floor is kept at a low relative level, viz, R L 138, or 9 feet below the level of cill of the canal head, in order to form a silt settling basin, to be periodically scoured out by raising the regulator gates in imitation probably of the corresponding data of the Rupar Head Works of the Sirhind Canal (see Fig 1, Chap VIII) The cill level of the canal head is at R L 147 00, thus a depth of no less than 9 feet of sand can accumulate in front of the canal head, and even more, if double draw gates were adopted But experience at Rupar has shown that as soon as the sand has risen to within 5 or 6 feet of cill level of canal head, it begins to pass over the cill and into the canal When that happens the canal head is closed and the weir sluices are opened for scour

At Laguna the water level above the weir, of the maximum flood, is at R L 156 000, or 18 feet above, the weir sluice sill at 138 The high flood level at Rupar is 875, or 18 feet above floor of weir sluices, which is at 857 The maximum flood of the Colorado at Laguna is passed with an afflux of 5 feet That of the Sutlej at Rupar has an afflux of 2 feet The Rupar system of silt-control is open to the serious objection that it necessitates periodical closing of the canal and consequent damage to agriculture It is not practised anywhere in the Punjab, except at Rupar Elsewhere the river is allowed to flow freely through the weir sluices, so as to keep the river bed scoured deep in front of the canal head

(29) The Madhopur Weir sluices of the Bari Doab Canal, a print from a photograph of which is given as Fig 14, form an instructive example of

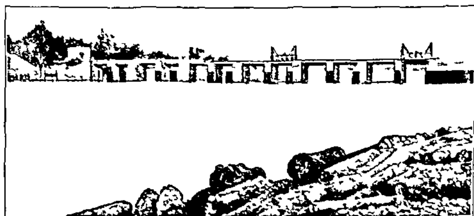


Fig 14 —Weir Sluices Madhopur Bari Doab Canal Up stream 20 feet spans Double gates.

remodelling an old work on modern lines The sluices originally consisted of a series of 5 foot vents These were demolished by floods and rebuilt with twelve vents, each 20 feet wide, with piers 5 feet thick between vents, with an arched bridgeway over all This type of sluices, 12 × 20 feet, has been the model from which the subsequently built sluices at Rupar, Khanki, etc, have been copied The pier noses and the abutments are faced with timber as a protection against damage by floating logs which, during floods,

float down from the Himalayan forests in vast numbers The 20-foot vents were supplied with double roller gates similar to those in the Chenab and Rupar Weir sluices, but gates of the " Stoney ' pattern are now in use

The view shows the two large travelling winches used in lifting the gates, standing on the tram line

(30) The thickness of the floor of a weir sluice head founded on sand is subject to kinetic as well as to hydrostatic considerations, and with regard to the latter, formula (3) of Chap VI , viz , $t = \frac{4}{3}\frac{H-h}{\rho - 1}$, will apply, whereas for the former, a heavy floor being a desideratum, the following empirical rule will be found in accordance with practice, viz —

$$t = \sqrt{\frac{3H}{2}} \quad . \quad . \quad . \quad . \quad . \qquad (3)$$

H being the statical head

(31) The term " Undersluice," familiar in Northern India, is in reality a misnomer, as in modern weir designs, " undersluices, literally as such, do not exist Weir sluices, or weir scouring sluices, is a more correct nomenclature

In the United States, the term *undersluice* applies only to weir or dam *body* sluices—as those in the Assuan or Bhatgarh dams

CHAPTER VIII

CANAL HEAD REGULATORS

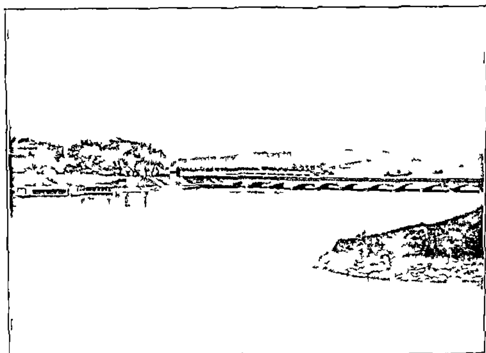


FIG 1 (*d*)—Head Regulator of the Sirhind Canal at Rupar

(1) THE function of canal head regulators or intakes is to adjust the admission of water into the canal from the river of supply, according to the requirements of the canal

In almost all cases this regulation is entire not partial that is to say on the one hand the full supply level in the canal cannot be allowed to be exceeded and on the other hand the head work should be capable of completely shutting off all supply even during the highest floods in the river From the above it will be evident that the pressure of the statical head of water to be provided may be much higher in this class of work than in weir sluices while the dynamical forces which are such potent factors in the latter are very much less in head regulators

(2) The following principles regulate the design of canal heads —

First The width of the openings should be as large as is consistent with easy manipulation of the draw gates

Secondly In order to prevent the entrance of sand into the canal which has caused much trouble and immense annual expense in clearance the

cill of the intake must be raised several feet above that of the adjoining weir sluices This is arranged for by constructing a crest wall across the entrance of the vents on top of which the lower gate rests or else by a still better arrangement the lower gate can be passed down behind the breast wall so as to act as a cill over which the water may flow by this device only surface water is tapped from the river At Rupar at the head of the Sirhind Canal the deep channel in front of the canal head is allowed to silt up and is then periodically scoured out by opening the gates of the weir sluices but this practice has not been adopted at any other canal head in the Punjab It involves the closing of the canal from time to time and this is prejudicial to agriculture

(3) The Sirhind Canal (Fig 1) was first opened for irrigation in the year 1882 and soon after experienced severe silt troubles So much sand was swept from the river into the canal and deposited on the bed of the latter as almost sufficed to put it out of action during the summer In the year 1891 about 36 000 000 cubic feet of sand were deposited in the first four miles of the canal making a shoal averaging 9 feet deep whereas the designed full water depth was only 9 6 feet Various remedies were applied but the one that proved successful was as follows The canal head was a bridgeway with thirteen spans of 21 feet divided by jack piers into thirty nine bays each five feet wide By knocking out the jack piers the waterway was increased from 195 feet to 273 At the same time a masonry wall 7 feet high was built across the entrance to the vents Behind this was arranged a movable cill gate 3 5 feet high so that the water could always be passed into the canal over the masonry cill at elevation 866 or over the gate cill at any level between 866 and 869 so as to skim off top water as free from sand as possible the paved floor of the river bed in front of canal head and weir sluices being at 857 The masonry weir crest was raised from 865 to 866 5 and on this were erected hinged shutters 6 feet high reaching up to 872 5 Whenever the canal was open the weir sluice gates were closed so that the current dropped its sand in the pocket in front of them When this shoal rose to within 5 or 6 feet of top of gate cill the canal was closed and the weir sluices opened and the shoal was scoured away through the latter

(4) The original design of the Chenab Canal Head at Khanki is shown in Fig 2 In this case also it was found necessary to knock out the jack piers under the main arches so as to increase width of waterway from 6 5 × 3 × 12 = 234 feet to 24 5 × 12 = 294 feet and also to raise the masonry cill from elevation 717 to 721 or 6 feet above the floor of the weir sluices and to furnish it with a movable gate cill capable of rising up to 725 An additional canal head was also built with twelve spans of 24 5 feet thereby increasing the total width of waterway to 441 feet The Rupar system of silt exclusion was tried but it was found inapplicable to local conditions Silt trouble persisted and it was found necessary to build up the masonry crest of the weir 4 feet higher *i e* up to 7 6 5 with 6 foot shutters on top of that again,

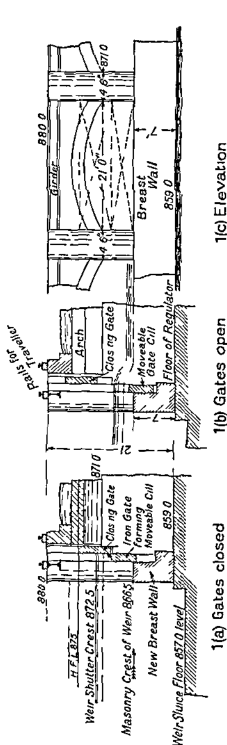


Fig 1 Sirhind Canal Head Rupar as remodelled

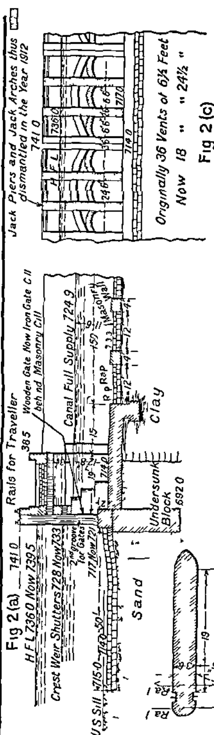
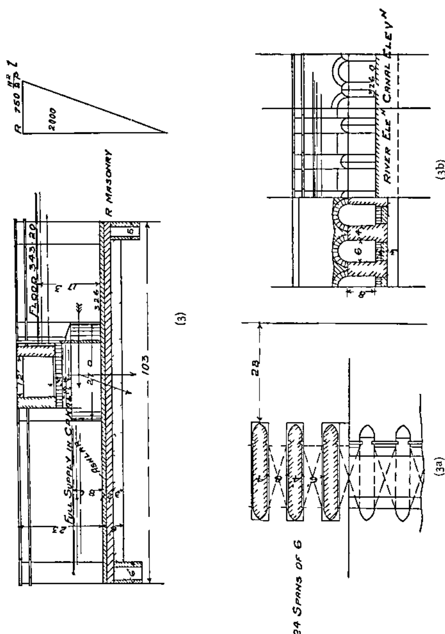


Fig 2 —Chenab Canal Head at Khanki, as remodelled

and to furnish the canal head with a new movable cill capable of rising up to elevation 727 The 3 tier gates of the weir sluices were scrapped and replaced by single gates with " Stoney " live rollers

Canal intakes are generally built on a clay or loam foundation, so that although subjected to a much greater hydrostatic pressure than is the case with weir sluice floors the pressure generally stops short of the beginning of the floor This is evidently the case here (Fig 2) The



FIGS 3 3a 3b —Son Canal Head at Dehri

head is 22 feet and with $c = 15$ l will be $15 \times 22 = 330$ feet The actual length of percolation provided is but 94 feet consequently the head work must rest partly on a clay substratum In such cases the hydraulic gradient will be horizontal and the pressure area a rectangle as there is no outlet for percolation

(5) The Sôn Canal Head at Dehri (Fig 3) is situated on a river which carries an immense quantity of heavy sand in suspension

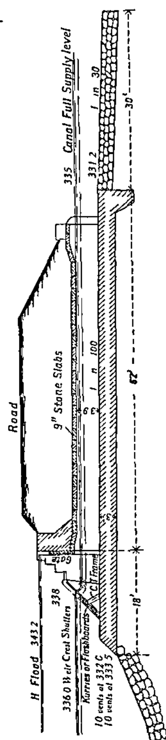


Fig 4 —Surface Supply Inlet Sôn Canal

The head regulator was located at the bottom of a deep quadrant bay, some distance from the undersluices, and the pier noses are recessed 30 feet behind the weir sluice land abutment This is shown on the block plan, Fig 8a, Chap VII

The cill of the canal head is flush with that of the weir sluice This arrangement, as might well be imagined, was not satisfactory, regarded from the point of view of silt exclusion Great masses of sand were yearly washed into the canals at both sides of the weir, necessitating the maintenance of a fleet of steam dredgers in the attempt to keep the canal open at all This nuisance continued for some five-and-twenty years, until at last the Rupar method of silt control was adopted, viz, a raised cill to the intake, and the closure of the gates of their weir sluice as much as possible, so as to allow of still water in the deep channel, whereby deposit outside the canal head is encouraged

(6) The canal head waterway was increased by construction of a supplementary head, to be used only at times of high supply in the river in conjunction with the old head, a temporary raised cill being formed in the latter by keeping planks in the grooves below the gates which are single and of antiquated pattern Whenever the river water level rises above 336 5 (½ foot above top of weir shutters), the wooden cill of the canal head, composed of kurries, must never be lower than 333 5 In this way only top water is taken into the canals, and when the river rises above 338 0 the canal heads are closed altogether

Fig 4 is the section of the supplementary surface supply inlet It consists of twenty small openings roofed by slabs, outside which are cast-iron inclined frames, on the sloping sides of which "kurries" or flashboards are placed up to any height which may be desired, and at time of floods the vents are closed by drop gates, and the regulating planks removed The raised cills or crests of the two works are kept at the same level

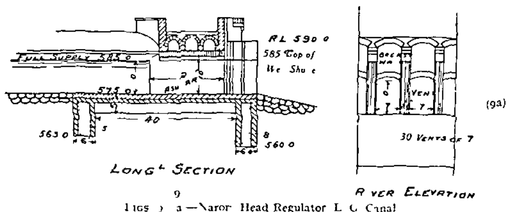
FIG 4a —Surface Inlet Sôn Canal

The photograph Fig 4a is derived with others by permission from The Irrigation Works of India

(7) The head regulator of the Son Canal is of the so called **U** pattern having a depressed arch springing at canal full supply level over which are two spandrel walls the space between being filled up with rubble stone or concrete

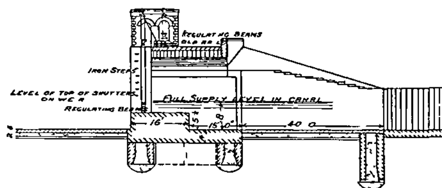
This work is also partly if not wholly on a clay foundation The piers are much too thick being nearly 7 S

(10) The Narora Head Work (Fig 9) was the first work supplied with *gates fitted with rollers or rather wheels the axles of which work in bearings*



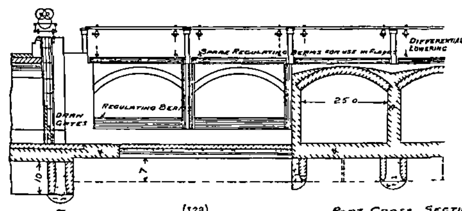
Figs 9 9a —Naror Head Regulator L G Canal

There is no disadvantage in this, particularly at this site, where the canal is in rock cutting, a small extension of the floor and wings being all that can be necessary It is suggested that this arrangement of double sluices, which, as far as is known, has hitherto not been tried,* is worth of adoption in all head works where the river in flood is very deep and where silt deposit gives trouble The silt in suspension could be allowed to accumulate in front of the head work to a considerable depth, and then by opening the weir sluice



LONG^L SECTION THRO HEAD

(12)



(12b) (12a) PART CROSS SECTION

LONG^L SECTION THRO UNDERSLUICES OF CANAL HEAD

FIGS 12 12a 12b —Jamrao Canal Head.

gates it could be swept out, leaving a clear channel ready for the time when the water level has fallen, and supply is drawn from the lower gates

With the adoption of roller gates, the lifting apparatus can consist of a travelling winch spanning the groove openings, the external arching being provided for that purpose In the existing work, screw lifting gear is used, and that of a primitive pattern, the female screw, held in collars, having the motive power applied, while the male screw rod attached to the gate rises

* But see Fig 13 par (15) Treble tiers of vents had already been applied to the head of the Trebeni Canal in the year 1901 Also to the head of the Upper Jhelum Canal at Mangla in 1908 Fig 11 shows only two tiers of vents but Fig 11a shows three tiers —ED

through it These are worked with great difficulty The absence of roller attachment necessitates the use of screws to force the gates down against the water pressure, whereas where rollers are provided the gates drop into place of their own weight and only require gear for raising

(13) Fig 12 shows the Jamrao Canal Head off-taking from the River Nara, an old channel of the Indus River

In this design the canal bed is at the same level as the floor of the weir sluices Across the head a breast wall is built 5 feet high The canal full supply is 8 feet deep and the crest of the weir shutters is 10 feet above floor, consequently 5 feet of water can pass over the raised cill crest, which will afford the supply required in the canal

For the purpose of diminishing end contraction in the submerged film passing the dwarf weir, the openings have been made very wide, viz, 25 feet spans, of which there are six The springing of the arches is at maximum flood level, *i e*, at 15 feet above the canal bed, the closure of the 10 feet space above the cill is effected by baulks, or regulating beams, lowered into grooves and raised by means of differential tackle The weir sluices shown in section in Fig 12b consist of seven spans of 20 feet, total ventage, 140 feet against 150 feet of the canal head The partial regulation of the weir sluice is up to a depth of 10 feet, and is effected by three draw gates manipulated apparently by fixed winches at each pier head

(15) In Fig 13 are represented plans of the Trebeni Canal Head The Trebeni Canal, which is situated in the province of Bihar and Orissa takes out of the Gunduk River in its upper reaches The design of the work is of the usual heavy Bengal pattern of the U shaped type added to which, however is a novel device for the purpose of tapping surface water at different levels of the river with the object of preventing or at least ameliorating, the entry of silt into the canal This is effected by the addition of two tiers of arches built towards the river These vaults are closed by baulks let into grooves so that the water can be admitted over them at three different levels

The waterway is designed so that full canal supply can be obtained with only 2 feet depth passing over the platforms The water when admitted, falls into the cistern formed in front of the sluice proper which latter can be closed by a gate operated by a screw worked by a capstan head In place of the breast wall, now usually adopted to raise the cill of a canal head the whole floor of the regulator is raised 3 feet above the canal bed which level is reached by a pitched slope succeeding the masonry floor This is shown in the transverse section of Fig 13

(16) The principle of the design is much the same as that of the Betwa Canal Head (Fig 10) It no doubt answers its purpose very well but the question naturally arises whether the equally good results could not be obtained at a much less expenditure The section of the work is to the eye exceptionally heavy even for the great head of 26 feet of water which it has to withstand The only way to settle this question is by an alternative

There is no disadvantage in this particularly at this site where the canal is in rock cutting a small extension of the floor and wings being all that can be necessary It is suggested that this arrangement of double sluices which, as far as is known has hitherto not been tried * is worth of adoption in all head works where the river in flood is very deep and where silt deposit gives trouble The silt in suspension could be allowed to accumulate in front of the head work to a considerable depth and then by opening the weir sluice

LONG SECTION THRO HEAD

(12)

(12b) (12a) PART CROSS SECTION

LONG^L SECTION THRO UNDERSLUICES OF CANAL HEAD

FIGS 12 12a 12b—Jamrao Canal Head

gates it could be swept out leaving a clear channel ready for the time when the water level has fallen and supply is drawn from the lower gates

With the adoption of roller gates the lifting apparatus can consist of a travelling winch spanning the groove openings the external arching being provided for that purpose In the existing work screw lifting gear is used and that of a primitive pattern the female screw held in collars having the motive power applied while the male screw rod attached to the gate rises

* But see Fig 13 par (15) Treble tiers of vents had already been applied to the head of the Trebeni Canal in the year 1901 Also to the head of the Upper Jhelum Canal at Mangla in 1908 Fig 11 shows only two tiers of vents but Fig 11a shows three tiers—ED

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FIG 13 —Trebeni Canal Head on Gunduk River

design, which is furnished in Fig 14 In this, the alternative design (Fig 11), has been followed, but with the provision of three vents, one above the other, in each bay The spans are widened to 10 feet the thickness of the piers being retained at 4 feet The platforms situated above the arches are made deep

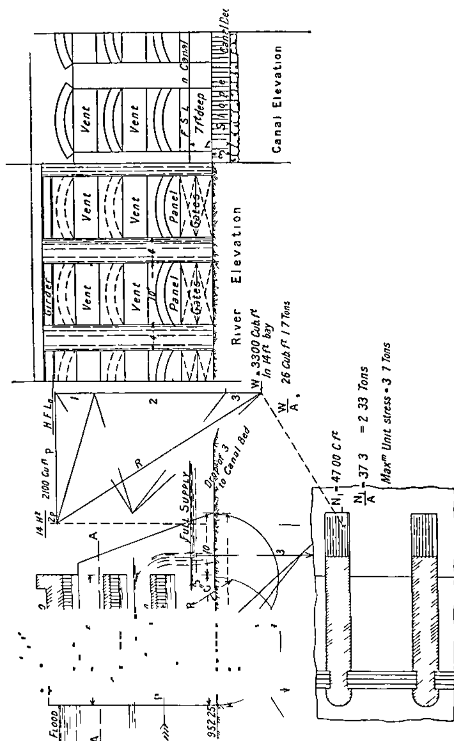


Fig 14 Trebeni Canal Head Alternative Design

enough to accommodate the drop gates of which there are four each working in a separate groove Two a e provided in the bottom sluice way so as to enable the upper gate to be either lowered behind the bottom one or raised above it without blocking the vent In the section the bottom

gates are shown closed, the second one is partially open, admitting water, while the third, which the water has not yet reached, reposes in the space provided for it below its vent. This arrangement permits of easy regulation of the water at any level—only surface water being drawn The water falls over on to the floor of the regulator beyond the arches There can be no reasonable objection to this, the floor, if properly built, will stand a vertical fall without the least cause for anxiety, although the firmly-rooted objection to vertical overfalls without the provision of water cushions is an old long-exploded prejudice which still lingers on The stress diagram and the reciprocal lines of pressure on the section prove that the centre of pressure falls 2 feet within the middle-third of the base

The maximum unit pressure in the masonry of the pier works out to 3 7 tons—a quite moderate amount The following comparative statements of the cubical contents per foot run show a saving of 40 per cent in the alternative over the original design —

	Contents in one bay	Length	Per foot run
Original section - -	3856	10′	385
Revised ,, - -	3300	14′	236

(17) Almost all canals take off at right angles to the axis of the parent stream, *i e*, generally parallel to the direction of the weir As the course of the canal must be more or less parallel to the river, this arrangement involves a large curve in the canal excavation close to the head work This is objectionable as causing a diminution of the velocity of the current, with consequent increased liability to the deposit of silt The Kistna East Canal, Madras Presidency (Fig 28b, Chap VI) takes off at a very sharp angle to the axis of the stream, probably in order to avoid a rocky hill or some such impediment The difficulty in that case is got over by the head regulator (which is "on the square") discharging into a large basin from which the canal takes out A similar difficulty is provided for by the arrangement shown in Fig 13a, Chap VII, of the Laguna Head Works

It is advantageous to construct skew heads at some canal head sites The disadvantage which is inherent in a skew head is the widening of the opening involved and consequent increase in the length of the gates Now, however, that anti-frictional rollers can be fitted to any gates, be they iron or wood, thus immensely reducing the power required to manipulate them, this objection vanishes altogether There remains only the question of cost, the skew head being naturally somewhat more expensive

In Fig 15 a design for a skew head is exhibited The angle of a skew is 60 degrees

(18) If the floods in the stream or river are not very high, a regulator with gates at the canal head can be dispensed with altogether, and the

canal intake will then consist simply of an opening protected by masonry flank walls and floor This arrangement is common in American head works

(19) The majority of rivers and streams in Burma from which water for irrigation can be taken are not, as in the case of Upper India, large rivers with

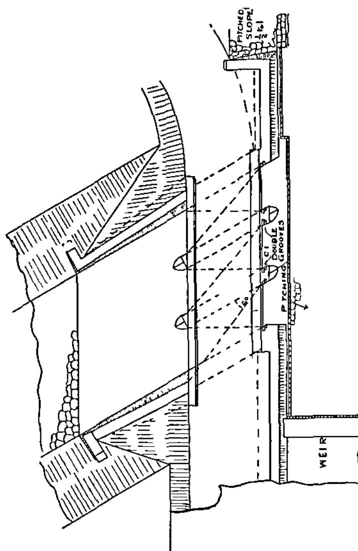
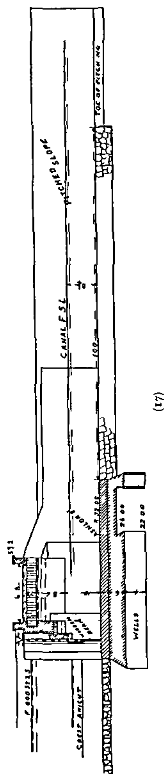
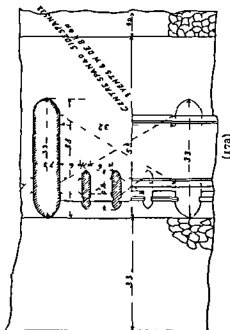


Fig 15 —Plan of Skew Head

perennial flow, in many cases derived from the melting of snow on the Himalayas but are intermittent in flow coming down in occasional freshets during the rainy season and running perfectly dry during most of the rest of the year Such streams are numerous in most sub tropical countries It is a common error to suppose that small rivers that only bring down water to local rainfall are useless for irrigation purposes such is by no means the case Where the local rainfall is insufficient to raise valuable crops such as





FIGS 17 17a 17b—Dauteshwram Head Regulator Godaveri Canals

rice or sugar cane it can be supplemented with great advantage by irrigation from an intermittent stream Besides which, these rivers often come down

in flood when there is no local rainfall a rainburst in the hills where their source lies causing a freshet The water can be utilised either by direct

(18b)

(18)

(18a)

Figs 18 18a 18b—Batarlanka Head Regulator

irrigation from a canal or a combination of direct irrigation and storage. The author has constructed several works of this description in Upper

Burma First, the stream was dammed and water taken off by a canal, from this direct irrigation on fields took place, and surplus water was allowed to pass through the fields, which were under a rice crop, and tail into another drainage line This again was dammed, and another small canal taken off In this way several drainage systems were practically connected, the main stream tailing into a large tank, whence the surplus water over the waste weir found its way into other tanks and eventually into the original watercourse, which again was dammed and the procedure repeated The fields themselves particularly rice fields which are banked by mud walls 12 inches high, will hold an immense deal of water, and consequently do not require a continuous supply, a freshet once a week or ten days would keep them well supplied with water The same applies to other crops, but in a less degree

(20) The thickness and length of the floor in the examples given are designed on the assumption that the foundation is sand The head regulators of canals and the weir sluices are often founded on clay In such cases no special calculation is required for the stability of the floor with regard to upward hydrostatic pressure The thickness of the floor may be determined by the engineer according to his judgment of general considerations

(21) We will now give two examples of the old style of canal head sluice, of which there are many examples in Upper India, as well as in Madras The design consists of a large span bridge with the interior filled up with smaller piers and vaulting The system, now that larger vents can be adopted, is quite obsolete Fig 17 represents the Dauleshwiram, and Fig 18 the Babarlanka Canal Head The works are situated on either side of the Godaveri Anicut These plans are instructive mainly as regards the foundations, which must be partly on clay

(22) An example of a canal head work built on boulder formation viz, that of the Western Jumna Canal Head, is given in Fig 19 In this case

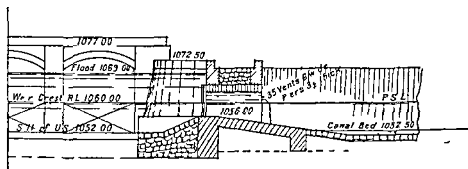


Fig 19 —Western Jumna Canal Head

the cill is raised 4 feet above that of the weir sluice, the floor being built on a slope forming a rapid On this sloping floor the piers of the work are

built, a rapid of this description discharges just as much as a vertical drop The object of raising the cill was to keep boulders from being carried into the canal but owing to the fact that the weir is merely a pavement laid with top flush with river bed there is nothing to ensure scour or to prevent shoaling in front of the canal head and when the river is in flood if the canal head happens to be open the noise and vibration of boulders being rolled through canal head can be distinctly heard and felt To prevent this shoaling a raising of the river weir and of canal head cill seems necessary

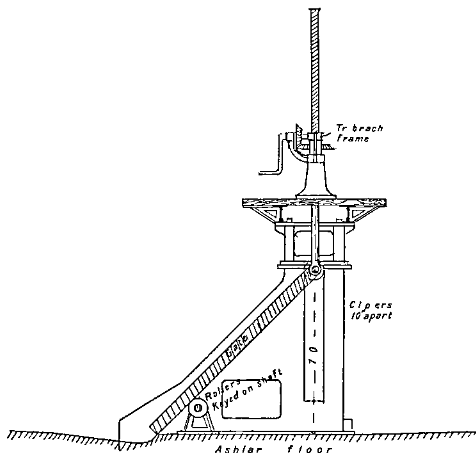


FIG 20—Goulburn Canal Regulating Gate

The vents which are only 6 feet wide are closed by wooden draw gates a recess being formed by an upright beam built into the cill behind which the gates work This system of narrow spans is now quite obsolete wider spans closed by double roller gates raised by a travelling winch being recognised as preferable

Plans of the weir sluices of this head work are given in Fig 12 Chap VII Eight of the spans farthest from the canal head have been closed with a wall up to R L 1060 in order to concentrate scour in front of canal head and to prevent shoaling there Such scour would be facilitated if the crest of the masonry weir were raised higher

(23) Fig 20 illustrates a remarkable and novel design for canal regulating head and gates, employed in the Goulburn Canal, Australia, which is given in "Irrigation Engineering" (Wilson) As will be seen, the gate, when closed, is inclined at 45 degrees and is attached at the centre to the end of a screw rod, which moves it in a vertical direction The other end rests loosely near the centre of pressure on four rollers carried on a shaft, which is supported by four fixed plummer blocks The upper ends of the gate are carried on wheels which just fit in vertical grooves in the framework of the iron piers The gate is thus forced to work vertically, the bottom end is consequently pushed downwards and outwards, eventually assuming a horizontal position when the sluice way is completely open By this ingenious disposition regulation can be effected with a minimum of applied force, and, in addition, the water is only drawn from the surface. This arrangement, however, would not work where silt is allowed to accumulate at the foot of the gate, and consequently would not answer for head regulators in ordinary cases

The piers, as well as the abutments, are built of cast iron, and are 10 feet apart

(24) The screw-lifting apparatus consists of a cast-iron standard provided with a separate head, carried on an oblong vertical frame of two arms joined at the head In the space thus formed, the female screw head fits, to which *is attached a horizontal bevel wheel This bevel wheel is worked by a* vertical pinion attached to handle shaft, which is carried by a third independent arm The hollow screw head bears upwards on the frame head and downwards on the top surface of the lower part of the frame, hence does not require a separate thrust plate

This arrangement, in common with all others where the female screw has the motive power, is subject to the great disadvantage of the screw rod being in one piece and rising above the platform and frame, and being thus exposed to dust or rain, besides being liable to be bent or damaged As fully explained in Chap XIV, where several examples of screw gearing are given, this fault can be remedied by having the motive power applied to the solid, not to the hollow screw The latter, which should be fixed to the head of a pipe, is prevented from revolving by being attached to the gate, as also to guides, and thus is forced upward or downward by the male screw, which travels inside the pipe Thus the solid screw is effectually protected from dust and water, and nothing appears above the head of the standard

This arrangement is obviously superior to any other, and markedly so where the lift is short, as the expensive solid steel screw need only be as long as the lift of the gate, whereas the long connection is formed by the pipe, which is better suited for compressive and tortional strain, and need not be more than an ordinary gas or water pipe having a hollow screw head fitted at one end Ball bearings should also be fitted to the thrust collars

The propriety of using cast-iron frames for piers and abutments of regulators in lieu of masonry is entirely one of comparative cost, which varies immensely in different countries, and so cannot be settled by any hard and fast rules.

(25) In Fig 21 we have an instructive example of recent American practice viz that of the Minidoka Canal Head in Idaho Most irrigation works in the United States have hitherto been constructed in a large measure of timber so that the institution of permanent works of masonry is a comparatively new departure Under these circumstances it is disappointing not to find in these latest productions no improvement over the Oriental types common to India and Egypt but on the contrary discarded forms are reproduced The principle of modern improvements largely influencing design is the adoption of anti friction rollers to draw gates whereby larger vents can be used than were formerly practicable which arrangement tends to economy of material as well as to the possible avoidance of expensive

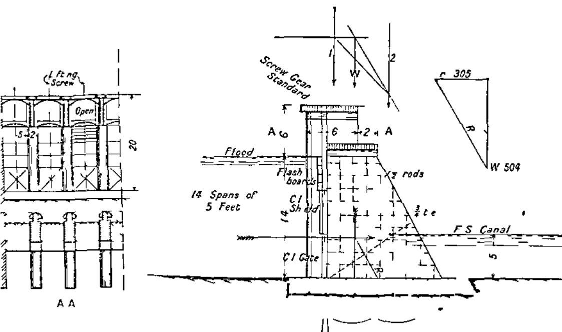


FIG 21 Minidoka Canal Head

screw gear for manipulating the gates and also the practice of admitting surface water only

In this work the head of water to which the superstructure is exposed is moderate being only 14 feet

The closure of the vents is effected by a cast iron draw gate working in the grooves of a high frame similar to that used in reservoir sluices The upper part of the frame is provided with a cast iron shield behind which the gate is drawn when lifted This shield takes the place of the breast wall or depressed arch usual in Indian works Above the shield closure is effected by baulks or flash boards as they are termed in America This arrangement enables water to be drawn from the surface during high floods

In the design the piers are 2 feet thick to spans of 5 feet a proportion of 4S

The resultant line of pressure, together with the concomitant graphical diagrams, has been drawn on the profile in Fig 21 The pier bases are subjected to a very moderate maximum pressure estimated at under 1½ tons per square foot In spite of this the piers are reinforced with a network of steel bars which, as well as the anchor bars let into the rock, seem rather unnecessary

The adoption of a single lifting gate is open to the objection * that water can only be admitted at the bottom, i e, under the gate

The screws working the gates are operated by a worm and bevel gearing, which is attached to a hollow or female screw head, held in the standards

Fig 23 —Folsom Canal Head

This revolves and draws the long solid screw bar to which the gate is attached up and down as may be required When the gates are raised these fourteen solid screws will project 10 feet above their standards into the air This objectionable arrangement is avoidable, by the simple expedient of applying the motive power to the solid rod, while the female screw head is attached to the end of a pipe fixed to the gate, which pipe does not revolve, and inside which the solid screw is protected from wet or rust, nothing appearing

* If a masonry wall ... feet high ... the grooves ... to act as gate this as well as gates for closure —Ed

above the standard The plans are derived from Irrigation Engineering (Wilson)

(27) In Fig 23 is another example of a recent work in America viz, the Folsom Canal Head Regulator This work consists of three spans of 16 feet, the piers being 6 feet thick or a proportion of 4 *S* nearly The head is much greater than in the former example being 32 feet In this design the depressed arch is very properly made use of its springing being 10 feet above floor The rest of the work is built up solid for an average width of 20 feet The area of each draw gate is 16 × 14 feet which is enormous considering the great head of water The gates are of wood not

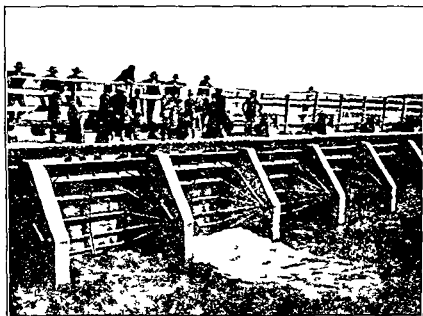


Fig 25 —Head Regulator of Twin Falls Canal

provided with anti friction rollers so that very exceptional arrangements have to be made to manipulate them This is effected by the adoption of hydraulic jacks one to each gate These jacks have plungers 14 feet in length fastened to the rear of each gate and are operated by a head of water provided from a power house

It is considered that the section which is very massive could be economised with advantage retaining existing conditions of span etc by reducing the weight of the solid mass filling and lengthening the piers The adoption of a U section with side breast walls filled between with any heavy material would effect great economy in masonry without much diminishing the weight a narrower top width with a battered inner breast wall being also adopted as shown by a dotted line on the section of Fig 21 As however in this canal in particular the prevention of silt deposit is an urgent matter it is deemed that a complete remodelling of the design allowing

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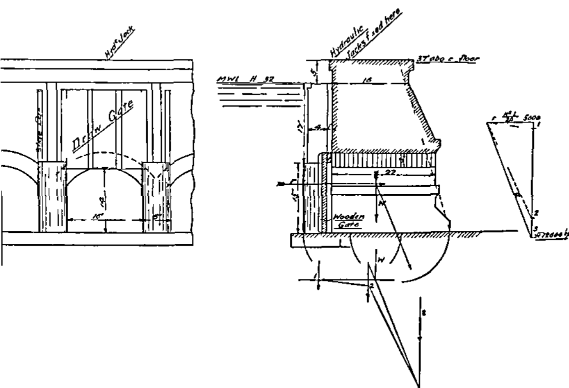


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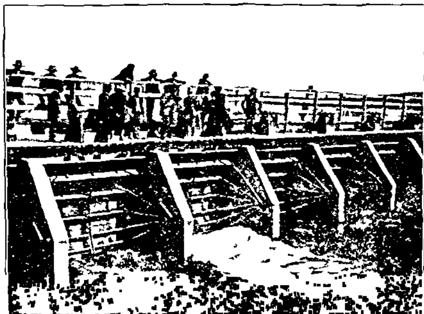


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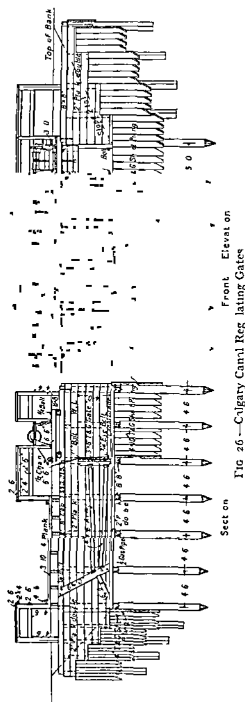


Fig 26 —Calgary Canal Regulating Gates

(28) In both these works, viz, Figs 21 and 23, the same arrangement is adopted for the gates which slide on flat iron bars let into the face of a recess in the piers, the single or double U shaped cast iron grooves usually adopted in modern practice in India and Egypt not being employed The rounded cut waters also do not extend above the height of the gates, so that there is no semblance of groove above this level This arrangement would not suit where gates are lifted at each end not at the centre, the groove being necessary to protect the lifting chains from the current

A site plan of the Folsom Canal Head Works is given in Chap XII

(29) It is unfortunate that very few drawings of American canal head works are available The pernicious modern system of substituting photographs for working drawings is responsible for this From these photographs a good idea of the style of the work is *obtained and* that is all

One of the Twin Falls Canal Head is reproduced by permission in Fig 25 It is taken from Schuyler's "Reservoirs" p 74 a technical book of the very highest class

From this, two main points of divergence from Indian head works is discernible First, the head of water is very low, *the gates are* 11 feet in height and there is not much in any way above them Secondly, there is no road bridge for cross communication for wheeled traffic This which is indispensable in a thickly populated country like India, can be

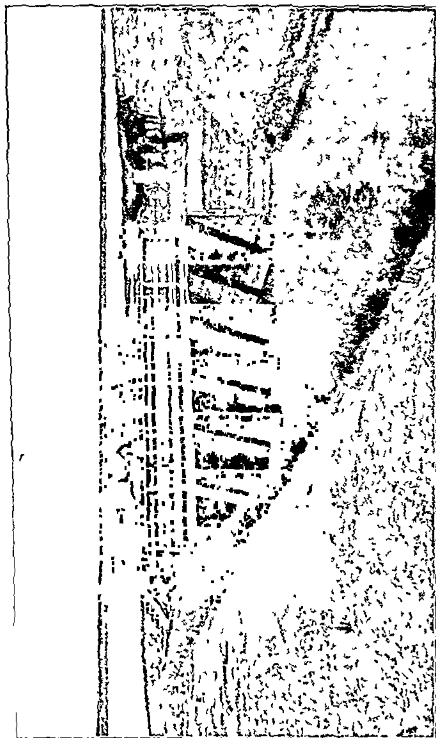


Fig 26a—View of a Calgary Canal Regulator

dispensed with here. The superstructure in this particular case consists solely of a wooden footbridge just sufficient to accommodate the man working the gates These latter gates consist of fan-shaped segments

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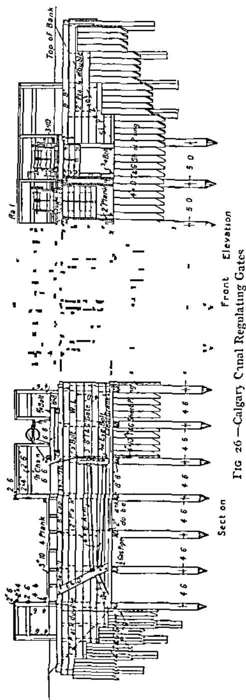


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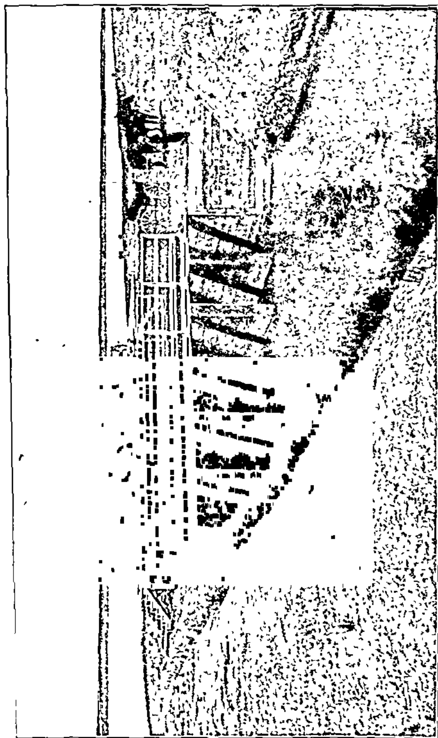


Fig. 26a.—View of a Calgary Canal Regulator.

dispensed with here. The superstructure in this particular case consists solely of a wooden footbridge just sufficient to accommodate the men working the gates. These latter gates consist of fan-shaped

pivoted in the concrete piers at some distance back This arrangement admits of easy manipulation *

Each gate has a fixed lifting apparatus, which probably consists of a drum revolved by gear which winds up a chain attached to the bottom of the segment Segmental gates of similar design were used in the Grand Barrage near Cairo On the remodelling of this work they were all removed, and double *drop gates in grooves with roller bearings, raised by a travelling winch,* were substituted As regards Oriental irrigation works, this style of water gate may be considered to be quite obsolete, the reason being that it has the fatal defect of only admitting water at the bottom When the gate is revolved the aperture below acts as a scouring sluice, the water entering under pressure as in a submerged orifice, and this must inevitably carry silt and sand into the canal This action in some cases, where levels admit of it, as in the Folsom Canal Head, is ameliorated by the adoption of under-floor sand traps But undoubtedly the better plan is to make the water enter over the tops of *movable gate cills or flash boards Where there is not much silt the Australian* regulating gate (Fig 20) would probably give good results and be just as easy to work by one man as the segmental gate These gates are 12 feet wide

The regulators on the new C P R Canal at Calgary, Alberta (Figs 26 and 26a), are fitted with wooden balance lever gates on the same principle These are centred very far back, so that the power to lift them is very small They have a very clumsy appearance

(30) In Upper Burma, south of Mandalay, an extensive system of irrigation from the Zawgyee and another river, or rather hill torrent, has



Fig 27 —Zawgyee Canal Head (Burma)

been in operation for 900 years These canals have recently been entirely remodelled on modern lines, and are now provided with head regulators escapes or spillways, falls, and all the various works incidental to proper regulation The head work of the Zawgyee Canal is illustrated in the

* The same design copied from a French model was once tried in the Punjab on a canal of 300 cusecs capacity but owing to the fact that water could only be admitted *under* the gates with great velocity drawing in excess of silt it was found advisable to scrap the arrangement and to substitute the ordinary style of sliding gate.—Ed

photograph in Fig 27 It consists apparently of small vents some 6 feet in width, closed by single wooden draw gates operated by separate fixed rack gear lifting apparatus Spare grooves are provided in front to allow of closure by baulks or flash boards, whenever necessary.

Although silt deposit is not troublesome in these canals, still double draw

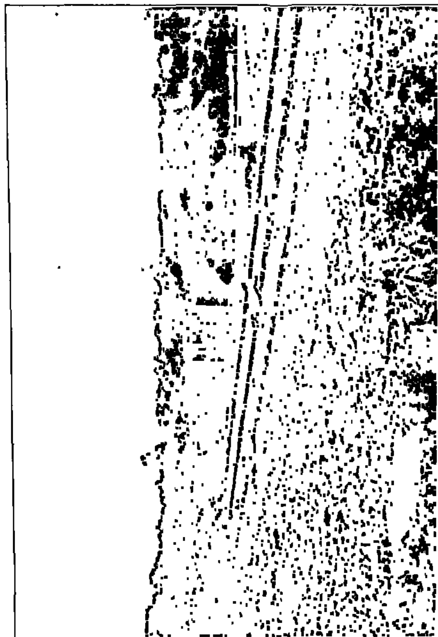


Fig 28 —Tennyetkon Canal Skew Head and Crib Weir (Burma)

gates would have been a better arrangement and more in accordance with the best modern practice

The weir was an old Burmese work of loose stone of type C, held in place by rows of small stakes These works required constant annual

repairs and even renewal; they are now probably sheathed with a covering of rubble masonry, or slabs on edge, and so rendered permanent. This treatment was proposed when the author was in charge of the Kyankse Irrigation Division in 1893, although at that time the remodelling project was but in embryo. This site is most picturesque; the engineer's inspection house close by used, however, to be jocularly termed a "promotion bungalow," on account of the extreme unhealthiness of the locality. The trestle work in front of the regulator prevents the entry of logs or detritus—it hides the weir crest from view.

(31) In Fig. 28 is a photograph of the Tennyetkon Canal Skew Head in Burma. This small work is situated above a crib weir, which has been

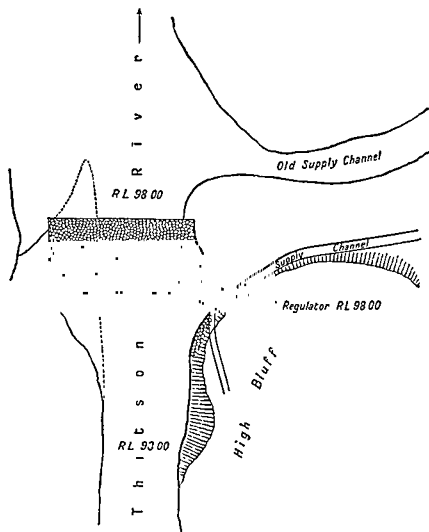


Fig. 28a —Site Plan, Tennyetkon Crib Weir.

illustrated in Fig. 19, Chap. VI. The head work is built high to admit of the draw gates being hauled up clear of the flood line, as, being an intermittent stream, its full flood depth has occasionally to be passed down. 28a is the site plan.

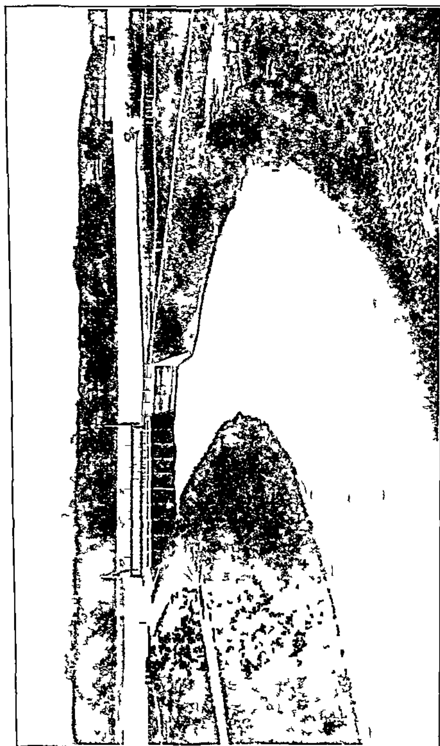


FIG 29 —View of Head Regulator Calgary Canal Alberta Canada Discharge 000 second feet

Fig 29 is a photographic view of the Calgary Canal Head on the Bow River near Calgary Alberta Canada

CHAPTER IX

CANAL FALLS

(1) When the slope of the ground surface exceeds the bed grade of a canal, vertical falls have necessarily to be constructed.

In many cases these drops in the bed are of frequent occurrence, and thus form a by no means inconsiderable item in the cost of the whole work.

A hundred years ago, when the first great irrigation works in India, such as the Jumna Canals, were constructed, hydraulic science was in a less advanced state than at present and successful precedent for guidance in design hardly existed. On these canals no masonry falls were built at first, nor until erosion of canal bed rendered such construction necessary. During the period 1840–50 the drop walls of falls on the Ganges Canal were designed on the profile of an ogee curve. The object sought to be attained was that of diverting the falling current smoothly down the descent, and by this means obviating any direct vertical impact in the floor. The masonry of these falls, however, happened to be generally of poor material, and it was severely damaged, especially where standing waves formed upon the floor, and as quick setting cement was unknown in India in those days repairs were ineffective. On the Bari Doab Canal, "rapids," or falls with descents sloped at 1 in 10, or 1 in 15, were successfully adopted but elsewhere vertical drop falls with cisterns, or water cushions below, came more generally into favour.

Up to a comparatively recent date the crests of the drop walls of canal falls were built raised above the canal bed level of the upper reach. This effected a vertical contraction of the area of the waterway at the fall itself, proportionate to the increased velocity of the current, and thus the velocity of the overfall was restrained to that admissible in the earthen channel above; that is to say, the velocity of approach was restrained to that of the supply channel.

For example. Supposing the water in the upper reach to be 6 feet deep, with a mean velocity of 3 feet per second, the mean discharge per unit width of channel would be 18 cubic feet per second.

If the crest of the weir were built level with the canal bed upstream, the length of the crest being that of the average width of the channel, then the depth of film passing over will be such as would suffice to give a discharge of 18 cubic feet per second. Now, the formula for discharge over a weir (free overfall) usually adopted is $3.33d^{\frac{3}{2}}$; if this be equated with 18 cubic feet, or else the result obtained from tables, the value of d will be found to be 3 feet.*

* The mean velocity would increase as the current approached the fall, from 3 feet seconds to $\frac{18}{3} = 6$ feet seconds, and the acceleration of velocity in a decreasing depth of

If the crest of the drop wall were raised 3 feet above the canal bed a depth of film of 3 feet would, as we have seen exactly pass the discharge Hence the surface slope of the water of the canal up stream would in no way be affected, nor the mean velocity of the channel interfered with Similar effect can also be produced by lateral contraction but such contraction must increase from surface downwards to bed level

This simple matter of raising the crest of the canal weir would answer perfectly were the water level in the canal constant Such however, is not the case

This difficulty of varying supply is practically overcome by the device of so called notch falls which have proved successful to such a degree as to be now generally adopted in recently constructed Indian canals

In notch falls the crest of the drop wall is raised up to Full Supply Level, and the part thus raised above the canal bed grade is pierced by a series of trapezoidal openings termed notches through which the water passes The reduction in sectional area is thus effected by graduated lateral not vertical, contraction as the base of the notch is at the level of the canal bed up stream The subject of the discharge of canal notch falls has already been noticed in par (21) Chap V

(2) It now remains to undertake the practical design of the works in detail

In Fig 1 is given an elevation, a plan and a section of a notch opening said to be used in the Chenab Canal while in Figs 1a and 1b are similar views of a notch fall actually constructed on the same canal These drawings are derived from The Irrigation Works of India The elaboration of detail shown in the diagrams in Fig 1 does not * seem to be carried out in practice in Figs 1a and 1b in the latter case the profile of the notch sides on plan is simply a segment of a circle

The plan (Fig 1) would not answer practically on account of the space required between each notch opening to admit of the great splay given to the sides of the inner face Such elaboration would undoubtedly conduce to raising the value of the coefficient of discharge but considering that the trapezoidal notch is after all exactly suitable to the discharge at two levels only and the coefficient adopted is but approximate such extreme refinement in the design of the outline appears out of place

(3) The first point to be determined is the total top and bottom width required for the notches This will naturally indicate the length to be given to the crest of the fall between the abutments and the number of notches it is advisable to adopt †

water would be able to scour the bed up stream for miles This is what happened on the Ganges Canal but it was not the fault of the ogee drop It would have happened with a vertical-drop fall —ED

* The diagrams are not representative of orthodox Punjab canal practice See [illegible] Notes at end of this chapter Figs 1a and 1b represent the simpler types a[illegible] Distributaries or Rajbahas but Fig 1 does not represent the best modern [illegible] official types —ED

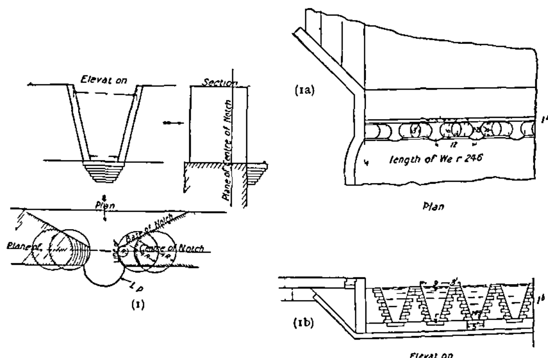
† Having determined the total top width and bottom width of the notches the [illegible] of notches may be decided in the light of convenience and the total width [illegible]

(4) The thickness of notch piers at notch cill level should be two thirds the water depth

The top length of the piers should not, for purposes of stability, be made less than their thickness, particularly in cases where the piers are carried higher than F S L, so that the fall can be crossed by planks laid on their summits, clear of the water

(5) A practical example of the method of calculating the number and dimensions of notches will now be given

The design in Fig 2 is based on an assumed canal discharge of 2,500 second feet with a mean velocity of 3 feet per second, and a full supply depth



FIGS 1 1a 1b

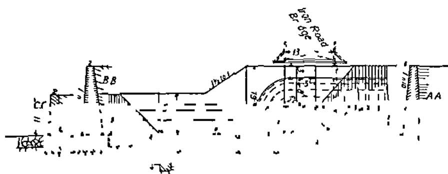
of 8 feet (free overfall) To produce these conditions the grade of the canal bed will be about ·18 per 1,000 feet

The area of the waterway will be $\frac{Q}{V}$ or $\frac{2{,}500}{3} = 833$ square feet

The width of the canal bed, with side slopes one to one will also be $\frac{A}{d} - d = \frac{833}{8} - 8 = 96$ feet The value of R is $\frac{A}{WP} = \frac{A}{b + 2d\sqrt{2}} = \frac{833}{96 + 22}$ $= 7{\cdot}06$ nearly

Now with regard to the number of notches The overfall being free, and the assumed velocity of approach above the average, it can be safely assumed

number will give the top and bottom width of each notch The spacing of the notches will be determined by the top width allotted to the notch piers in the light of convenience and structural appearance



(2b)

(2a)

Figs 2 2a 2b—Design of Notch Fall

Shown as 5 9" in Fig (2a)

that the top width of each notch will not exceed $\frac{3}{4}d$ or 6 feet We have seen that the length of the pier crests should not be less than half the depth

of water, or 4 feet, and as the piers are intended to carry a light bridge, this length might well be increased to 5 feet

The spacing of each notch will thus be 11 feet This will allow of eight notches, or a length of 88 feet between abutments, which is 8 feet less than the bed width of the canal As, however, the abutments will be designed with battering faces, the actual length at base of the notch piers has been made 85 feet

(6) Having tentatively decided on the number of notches, the discharge which each has to accommodate will be $\frac{2\,500}{8} = 312$ second-feet nearly

If reference is made to Table II Chap V, it will be seen that the discharge per foot run with $d = 8$ and $V = 3$ less 10 per cent, is 69 44 cubic feet per second The required half width of each notch opening (b) will then be $\frac{312}{69\,44} = 4\,49$ feet This is the value of the half width (b), whatever may be the slope of the sides of the notch opening

(7) The half width of the whole opening having been thus ascertained, we have now to fix the same for another lower depth of water

If a notch were designed to discharge precisely the proper amount at every change of level in the surface of the canal water in the upper reach, it would take an ovoidal outline similar to the profile of an egg with both ends truncated the thick end uppermost As however, it has been proved that a trapezoidal shaped opening correctly calculated to discharge the full supply and that due to one lower depth will answer all practical needs, this refinement need not be entertained

Orifices of this ovoidal section could easily be provided, were cast-iron piers adopted

(8) To revert to the lower depth, it is found convenient to place this level (d_2) below what is usually the lower supply in the canal (d_1)

In Upper India most canals run at two levels the higher when water is most in demand, *i e*, in the hot season, when the rivers rise owing to the melting of the snows in the Himalayas, and again a lower supply in the winter months, and this double supply level is what obtains in most canals

Owing however, to a variety of causes the fluctuations of level are often considerable, so that it is advisable to adopt a value for d_2 below that of the lower supply level d_1

This value should be a little over half the lower depth or $\frac{d_1}{2}$, or under half the full depth, *i e*, $\frac{d}{2}$

In the case we are considering, the lower supply level d_1 is assumed at 5 feet in depth, but the value of d_2 will be taken as 3 feet

(9) It now remains to calculate the discharge of the canal at this depth of 3 feet.

In this case $A = 99 \times 3 = 297$ square feet, $R = \frac{A}{WP} = \frac{297}{104\cdot4} = 2\cdot84$, S is given as $\cdot18$ per 1,000, or $\frac{1}{5{,}555}$.

We have now to estimate the mean velocity from the formula $100c\sqrt{RS} = V$.

Omitting c, the expression will become

$$V = \sqrt{2\cdot84 \times \frac{10{,}000}{5{,}555}} = \sqrt{\frac{5{,}540}{1{,}111}} = \sqrt{5\cdot11} = 2\cdot26.$$

If S per 1,000 be considered, the equation will be $V = \sqrt{2\cdot84 \times 10 \times \cdot18} = \sqrt{5\cdot11} = 2\cdot26$ (*vide* par (28), Chap V)

The value of the coefficient c, suitable to one of $R = 2\cdot84$ and of $S = \cdot18$ per 1,000, obtained from Table XII, Part IV, of the "Hydraulic Manual," is $\cdot7$ nearly The actual mean velocity will then be the previously obtained value of $2\cdot26 \times \cdot7 = 1\cdot582$ feet per second, and the discharge of the canal at the lower depth of 3 feet will be AV, or $297 \times 1\cdot582 = 475\cdot2$ second-feet

(10) Again dividing by the number of notches, the discharge through each must be $\frac{475\cdot2}{8} = 59\cdot4$ second-feet The half width b_2 is obtained, as previously, by reference to Table II, Chap V In this we find that the discharge per foot run, less 10 per cent with $d = 3$ and $V = 1\frac{1}{2}$, is 15·853 second-feet, b_2 will therefore $= \frac{59\cdot4}{15\cdot85} = 3\cdot75$ feet.

(11) Having thus obtained the widths of the notch openings at two known levels, viz, at $\frac{d}{2}$ and $\frac{d_2}{2}$, we are now in a position to calculate the top and bottom widths

The vertical distance apart of the two widths b and b_2, are $\frac{d}{2} - \frac{d_2}{2}$ The ratio of widening will then be $2\frac{b - b_2}{d - d_2}$ Let these horizontal and vertical differences be designated m and n Then the expression becomes $2\frac{m}{n}$ Now the top of the opening is distant $\frac{d}{2}$ above the position of b, and the base $\frac{d_2}{2}$ below that of b_2, consequently the top width will be $\left(\frac{2}{d} \times 2\frac{m}{n}\right) + b$, or $(b + d)\left(\frac{m}{n}\right)$ (1) Similarly the base width will be $(b_2 - d_2)\left(\frac{m}{n}\right)$ (2).

The value of m is $4\cdot49 - 3\cdot75 = \cdot74$
That of n is $8 - 3 = 5\cdot0$

Therefore $\frac{m}{n} = \frac{74}{5} = 15$, and transposing these values in the expressions (1) and (2) we have

top width $= 4\ 49 + (8 \times \cdot 15) = 5\ 69$ feet and
base ,, $= 3\ 75 - (3 \times 15) = 3\ 3$ feet

The average of both widths should naturally equal b, i e, $\frac{8\ 99}{2} = 4\ 49$

The size of the notch is made 5 feet 9 inches at top and 3 feet 3 inches at base With 11 feet centres, the top length of the piers will be 11 − 5 feet 9 inches = 5 feet 3 inches

(12) Supposing the fall to be not free, but submerged, i e, the drop in the canal surface to be 2 feet only

In such case, with full supply, the discharge (less 10 per cent), with $d = 8$, $V = 3$ and $H = 2$, as in Table IV, Series IV, is found to be 45 0 second-feet per foot run, the value of b will then be $\frac{312}{45} = 7\ 0$ feet instead of the 4 49 in the free overfall The lower depth d_2 will be submerged 1 foot, consequently its discharge by Table IV, Series II, deducting 10 per cent will be 10 6 and $b_2 = \frac{59\ 4}{10\ 6} = 5\ 6$

Then $m = 7\ 0 - 5\ 6 = 1\ 4$
and n as before $= 5\ 0$
Whence $\frac{m}{n} = \frac{1\ 4}{5} = 28$ and by (1) par (11)
the top width $= 7 + (8 \times 28) = 9\ 24$
by (2) the base , $= 5\ 6 - (3 \times 28) = 4\ 76$

Total 14 00 Mean, $7\ 0 = b$

With these dimensions, the yaw of the opening will be noticeably greater, and the spacing of the centres of the notches will have to be made $13\frac{1}{2}$ feet, the weir crest being lengthened accordingly

(17) In Fig 2 the weir wall is of the "hybrid" type of section (vide par (63) Chap II) The base being $\frac{H + d}{\rho}$ or, in Fig 2, $\frac{4}{9} \times 16 = 7\ 1$ feet (made $7\frac{1}{2}$ feet), in Fig 3, $\frac{4}{9} \times 11 = 5$ feet (made 5 feet 6 inches)

The top width is made 5 25 feet in Fig 2 The extra width is required for the accommodation of the notch piers The thickness of notch piers is 4 5 feet *

The length of the floor or apron is $2\ (H + d)$, according to the rule given in Chap IV par (20)

* As the Notch Piers have to sustain the pressure of water 8 feet deep flowing on either side of them with high velocity, their base thickness should not be less than $\frac{2}{3} \times 8$ = say 5 5 feet —Ed

The thickness of the floor is made $\sqrt{H + d}$ considered sufficient with a clay foundation

With regard to the width of the floor it is made somewhat greater than the canal bed width

The floor is terminated by a shallow curtain wall beyond which it is protected by pitching

(18) The abutments as well as the wings are all battered at 1 in 10 The abutments are continued up stream for a short distance parallel to the axis of the work forming the commencements of a pair of level crested splay walls with end returns

These not only form an excellent guide to the current on its contraction at the fall but with the assistance of the level crested portion of the dog legged down stream wings afford a wide level connecting bank and an approach to the bridgeway across the fall

(19) The down stream wings on plan are of the so called dog legged pattern having a re entering angle They start splaying outwards from the weir wall until the floor half widths are reached This portion is level crested When the floor widening is thus effected the direction of the base of the wings runs parallel to the axis of the work In Fig 2 the first portion has a sloping crest of 2 to 1 corresponding with the drop in the canal bank when this is overcome it continues in the same direction at the level of the down stream canal bank to the termination of the floor when another splayed return forms a junction with the canal bank side slopes

(21) The sections of the wings are strictly in accordance with the rules formulated in Chap I The sections at AA in Fig 2 of the up stream wings are of walls under partial earth pressure The earth pressure exists only as far as the up stream canal bed below this level no additional external force is applied so that the base at the upper canal level is made $4H$ 1 wide and below that point the back is carried down vertical but *the face continues battered as above ground* in addition a footing is provided near the base the 1 in 10 batter having at this depth to be thus supplemented to ensure the line of pressure falling within the base

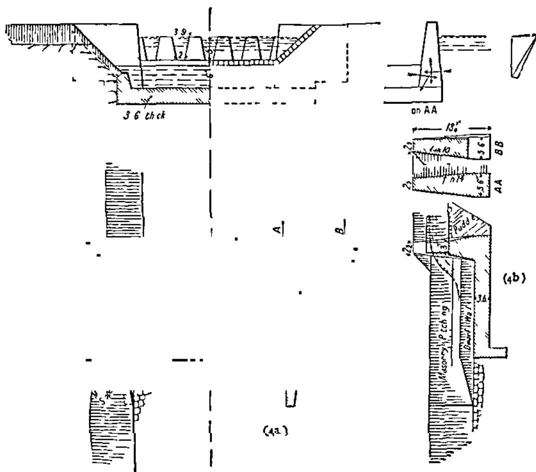
This important matter which is so often neglected has received con siderable attention in Chap I (*vide* pars (45)—(53))

(22) In Fig 2 arrangements are shown for a light bridge across the fall In this the notch piers are utilised to support square pillars of brick in cement which carry an I beam A second longitudinal is bolted to iron columns which are founded on masonry blocks situated at the general foundation level and these two are spanned by cross joists projecting well beyond the longitudinal beams which in turn are covered with planking Parapet rail standards are bolted to the projecting part of the joists and thus a 13 feet wide roadway is provided

The level of the platform is a little above that of the banks on either side, and the connection can be formed by a short slope

An arrangement of this kind is equally effective and vastly more economical than the heavy masonry over bridges which are such a common feature in old canal falls and which are perpetuated to the present day, as will be noticed later with reference to the Jamrao Canal Fall in Fig 5

(23) We now come to an altogether different type of construction for a



Figs 4 4a 4b —Notch Fall of ' Wingless Type

fall which is illustrated in Fig 4 The principle of the construction is to effect economy in the flank defences of the work by causing the earth to support the protective walls instead of *vice versâ* so that masonry pitched slopes take the place of the down stream flank retaining walls Up stream there are no water wings whatever Their place is taken by a horizontal and vertical prolongation of the weir wall forming single direct return flank walls on each side of the weir the up stream canal side slopes running right up to them

Down stream the width of the floor is terminated on each side by a dwarf flank wall 3 feet high, which runs out with level crest to the end of the floor,

whence the crest dips down at a slope of 2 to 1 to the down stream canal bed level

Above this dwarf wall a masonry slope rests on the side of the earth cutting at a slope of 1 to 1, and is carried on thus to the toe of the dwarf wall where it merges with the dry pitched canal slopes

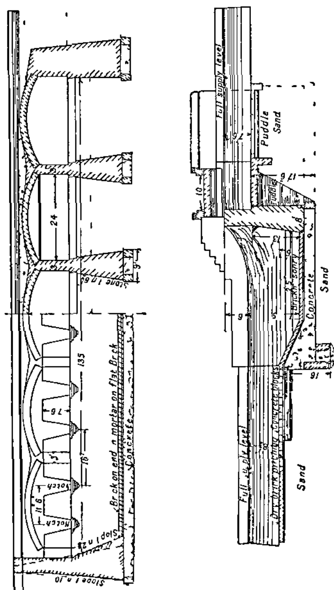


FIG 5—Fall on Jamrao Canal

The raised flank continuations of the weir wall have their foundations stepped up

These walls at their commencement are subjected to lateral water as well as earth pressure and so the section gradually diminishes in width till at their ends the earth pressure from the up stream side becomes very much reduced

This is explained by the half elevations and plans and the cross sections on AA and BB

(24) Excluding the weir wall and the visible portion of the floor the quantities of masonry in the flank defences are as follows —

Fig 4 2 210 cubic feet

Where falls are narrow deep and numerous this type of design is eminently suitable not only on canals but in reservoir bye washes canal escapes distributary falls etc The author has constructed several across rivers One of these is illustrated in Fig 24 Chap I and described in par (69) of the same chapter A photograph of it is given as Fig 13 at end of this chapter

(25) Fig 5 is an example of a recently constructed canal fall on the Jamrao Canal and is instructive as showing how a too close adherence to obsolete types results in a quite considerable waste of money

The plans are taken from the Min Pro Inst C E Vol CLVII p 278

The drop in the water surface is 6 feet whereas that in the canal bed is 7 feet the depth of full supply up stream being 7½ feet while that down stream is 8½ feet

(26) A cistern no less than 6 feet in depth is provided for a drop of nearly the same amount This is excessive Dyas formula is $x = H + \sqrt[3]{H}\sqrt{d}$ (see Molesworth s Pocket Book Water Cushion for Weirs) in which H — depth of water up stream of fall d = fall of water surface and x depth of floor of cistern below water surface down stream In our case in point x $H + \sqrt[3]{H}\sqrt{d} - 7\ 5 + \sqrt[3]{7\ 5}\sqrt{6} = 12\ 3$ feet Whence depth of cistern $12\ 3 - 8\ 5 = 3\ 8$ feet only

(27) A second point in the design which may be open to objection is the combination of an overbridge with the fall Except for reasons of economy there can be no possible object in combining these two works A road crossing for traffic purposes is not necessarily required at the exact site of a fall and a structure to serve both purposes may very possibly be located at a site which is not the best for either

From inspection of the drawing it is questionable whether the combination is productive of economy in the present case

The bridge consists of wide spans of 24 feet The foundations of the piers have to be carried right down to the base of that of the cistern *i e* to a depth of 18½ feet below canal bed level Were the bridge built as a separate work it might consist of moderate spans of say 15 feet with piers 8 feet high only resting on a floor platform which need not be more than 3½ feet thick enclosed between two lines of sheet piles The presence of the bridge piers on its crest necessitates the lengthening of the weir

Another point involving a further unnecessary lengthening of the weir is the excessive length of the notch piers which measure 6 feet long at the crest whereas 4½ feet or 5 feet would have been ample As there are ten

notches this adds another 15 feet to the length of the weir wall making it longer than is necessary

(28) The analysis of the section (Fig 5) with regard to l the length of enforced creep and to t the thickness of the floor is as follows —The maximum head occurs when the canal is nearly empty at which time only a small depth of water is passing down just sufficient to cause hydrostatic pressure but which will also fill the cistern The head will therefore equal the drop *i e* 7 feet The coefficient being 15 the value of l will be $7 \times 15 = 105$ feet The actual length of creep is about 110 feet so it is amply provided for in this respect

(30) Facility of method combined with certainty and precision are indispensable aids to successful design and the author s endeavour throughout this work is to smooth the rough paths and render the designing of works an easy matter

The question often arises in designing canal works as to whether it is more economical to provide one deep fall or several small ones As regards cost of masonry alone ignoring that of the earthwork in the cutting the advantage does not lie decidedly either way but the wider the fall the greater will be the advantage if any of the deeper fall the reason being that the cost of the flank defences bears a greater portion to the whole in a narrow than in a wide work In a long weir the main expense is in the weir wall and floor So ignoring the flank defences the sectional areas of the weir wall and floor alone for different depths of fall are given below In every case the value of d or depth of film is taken as 5 feet $H + d$ therefore will equal $H + 5$ in every case —

	Heights of Weir above Floor					
	Feet 5	Feet 7½	Feet 10	Feet 1[illegible]½	Feet 15	Feet [illegible]0
Sectional areas—						
Weir	19	45	53	[illegible]2	101	155
Floor	64	88	11[illegible]	14[illegible]	180	[illegible]50
Totals sup ft	83	133	1[illegible]0	[illegible]19	[illegible]81	405

	Sect area sup ft		Sect area in sup ft
Two [illegible]5 feet deep falls	1[illegible]6	Two [illegible] feet falls	[illegible]66
One 10	1[illegible]0	One 15	281
Three 5	49	Two 10	340
One 15	[illegible]81	One [illegible]0	403
Four 5	33		
One 20	403		

From the above it will be seen that the difference in quantities of weir plus floor is much the same in the case of several small falls or of one larger one the advantage generally lying with the smaller falls

The comparative areas of the flank walls will throw the balance further in favour of the smaller falls

In estimating the sectional areas of the weirs the base width is taken as that of the Hybrid section viz $\frac{H+d}{\rho}$ not $\frac{H+d}{\sqrt{\rho}}$ the thickness of the floor as $\sqrt{H+d}$ and the length 2 $(H + d)$

(31) In the dry zone of Western Canada in the Province of Alberta large canal systems are in process of formation the principal of which is the

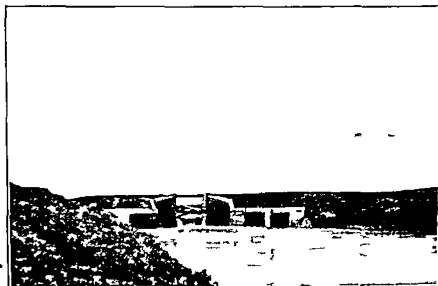
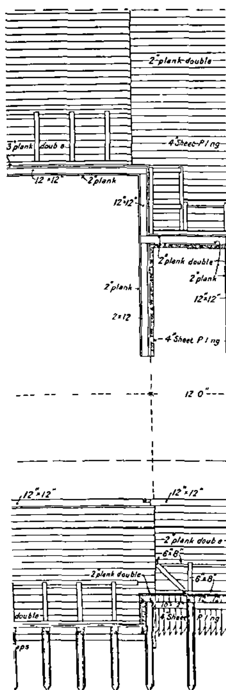


Fig 7a —10 Foot Fall on Calgary Canal Alberta

Calgary Canal undertaken by that enterprising corporation the Canadian Pacific Railway This canal a short description of which is given in Chap XIII will when fully developed be by far the largest on the American continent it is thus fit that the premier railway company of the whole world should also lead the record in irrigation enterprise The author has been privileged to obtain the blue prints of some of the principal works and it will be interesting and instructive to reproduce some plans particularly with regard to engineers in countries possessing timber forests in abundance where irrigation is practised In India itself timber is scarce and expensive but in Burma which has an extensive dry zone excellent timber hard wood as well as teak is available at moderate prices

The first illustration in Fig 7 is of a 10 feet drop situated 15 000 feet from the canal head The bed width here is 44 feet side slopes 2 to 1 the average width of the water section will therefore be 64 feet with a full supply depth of 10 feet



ection

Fig 7—10 Foot Fall on Calgary



In these falls the crest is not raised as is commonly the practice wherever notch falls are not adopted but on the contrary it is lowered 3 feet below the up stream canal bed grade the approach being in the nature of a rapid the crest itself is contracted to a width of 17 feet

The width of the floor below the drop 43 feet is nearly equal to the bed width A water cushion is provided 4 feet 8 inches deep

By the arrangement above sketched the overfall is concentrated in a narrow width and the velocity of approach largely augmented

The scouring action of the falling water must therefore be accentuated much above what it would be were the usual wide raised crest adopted

Modern practice in canal falls is in favour of notch falls in which type the current instead of being concentrated in one powerful volume is split up into several jets independent of each other However with a water section having an exceptionally low ratio of $\frac{\text{base}}{depth}$ as 4 4 together with a high velocity of approach of 3 2 feet per second the crest width of the fall should be greater than the bed width of the canal say more nearly equal to the average width of the water section which is 64 feet A photo of this work showing the cross bracing is given in Fig 7a

(32) The difficulty inherent in the case of wide canal works built of a so extremely unsuitable material as light pine wood is that the post and plank wing walls which support the earthen sides above and below the fall require strutting across for mutual support In the small Western American canals which are often not much larger than Indian distributaries this can easily be effected When however the same design is copied on a larger scale the cross ties have to be made of built beams and the expense is considerable This must be one reason why the wide portion is limited to the part below the drop wall

In this design the revetment walls of the water cushion well or pit are not shown as cross braced but this had eventually to be done as the pressure of 19 feet of earth would otherwise have soon overcome the slight resistance offered by the upright poles 5 feet apart

This difficulty in connection with wooden revetment walls is easily solved by simply stepping outside the trammels of precedent which seems to have such a hold on some engineers not only here but in India that is by abandoning the vertical walled lock or box style of construction of falls for an open one and of substituting the much more reasonable system of allowing the earth to stand on a slope instead of being held up vertically These slopes with the assistance of a little overlay of riprap will stand permanently at a much steeper inclination than the angle of repose of the material

This method of construction which is the same as that exemplified in Fig 4 will be found much cheaper the saving in woodwork being very considerable as a set-off against which is the cost of stone filling and of pitching banks and of a puddled clay wall in rear of the main cross drop wall which latter with wooden structures is deemed an essential feature This canal has the great advantage of a subsoil consisting of glacial detritus in which large

quantities of stones and gravel are mixed up with the clay Safeguarded from erosion by light crib work, stone forms an excellent and effective protection to earthen beds, or banks against water scour

(33) An alternative design on these lines is given in Fig 8 The width of crest adopted is 60 feet which dimension, if anything, errs in excess The up stream slopes are steepened from 2 to 1 to $1\frac{1}{2}$ to 1, at which they will easily stand with the possible addition of a 3-inch layer of loose stone The down stream side slopes are made such that they will intersect the wing crests at the same point as the up stream slopes, they will thus stand at ·87 to 1 *i e* a little steeper than 1 to 1 Considering that revetted slopes stand well at $\frac{1}{2}$ to 1 this inclination is by no means impracticable This slope is revetted with stone $2\frac{1}{2}$ feet thick at the base, tapering to 1 foot at the crest and the pitching is continued up to the end of the down-stream floor From this point down, it will widen out to the normal 2 to 1 The drop wall itself consists simply of a single wooden double-planked core wall

The planks are placed inside the posts, which are spaced 6 feet apart Another row of posts or piles is placed 15 feet in rear of the face wall, connected to it by sills at the top No other cross connection is permitted as otherwise leakage through the puddle wall, contained between the rows of posts would be encouraged, and the proper consolidation of the clay interfered with With regard to the wings both sides are planked, enclosing the puddle core This latter will be carried down in a trench to a depth of 2 feet below the top of the sheet piling which extends all along below floor level, stepping up in the wings The planking is double 3 inch, except in the rear of the wings where it is double 2-inch

The water cushion is omitted The greatest strain on the floor will be produced if 2 or 3 feet in depth were let down with the channel below empty, which eventuality is amply provided against by the cribbed riprap or filling The floor is composed of $4\frac{1}{2}$ feet of loose stone or rock, partly covered in by 3 inch planks spiked or screwed down to walings, which in turn are spiked sideways to the piles These latter are spaced in 6 feet intervals in either sense

The walings are not secured on top of the posts as sills, which is generally the best arrangement, but to the sides in order that they should form easy connection with the outside piles or posts of the drop wall

The last two rows of piles are raised 4 feet above the floor, to form a stop wall This will be filled up with loose rock, so as to allow free percolation, the stop wall has the effect of retaining a greater depth of water over the floor than is below it, and has been found to work well in practice The revetted side slopes are protected by planking spiked to walings, connecting the inner row of piles in the pit with a further row driven into the slope at a higher level This protection is only for a short distance, within the range of the falling water

Considering that canal works constructed of so perishable a material as soft wood can only be temporary, it is essential that they should be as economical as is consistent with safety The amount of material in Fig 7

(44 feet wide) is, piling 4,070 lineal feet, woodwork generally, 116,756 feet (board measure) (Board measure is cubic feet multiplied by 12)

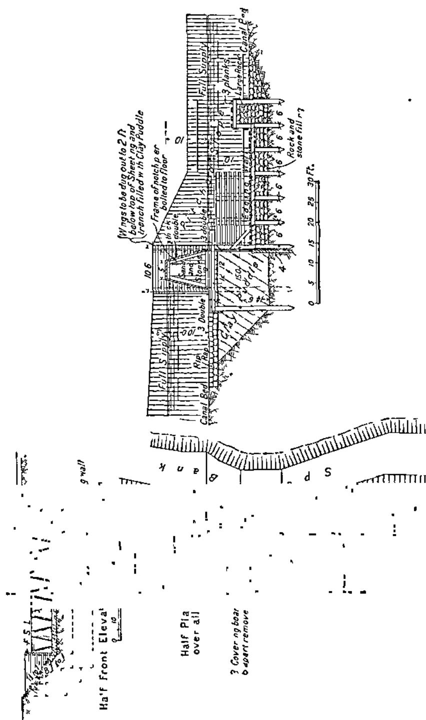


Fig 8 —Alternative Design (Wingless Type)

In Fig 8, with a 60 feet width of crest the corresponding quantities are piling 1,380, woodwork generally 46,400 B M , plus stone filling, rip-

rap and puddle Two drops of 5 feet would probably be cheaper than one of 10 feet and certainly much safer The notches it may be added are wooden frames covered with bent inch boarding caulked the hollow filled up with sand or small stones and the frame bolted down to the upper floor crest of the fall

The cost of the stonework cannot be estimated it will however not nearly equal that of the woodwork saved which costs $47 per 1 000 feet B M

(34) Another fall on this canal is exhibited in Figs 9 and 9a of an 11 feet drop in a secondary canal which has a bed width of 18 feet

This consists like the last of a narrow rapid followed by a laterally restricted fall but exaggerated It has the peculiarity of a kind of pen stock or pit at the head of the rapid the object of which is to hold up the water level and so limit the velocity of approach

It is considered *that this design would be improved if it consisted of* two distinct falls of 5½ feet on the same principles as Fig 8

This has been worked out in Fig 10

The quantities of woodwork are as follows —

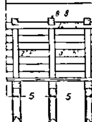
		Piles foot lineal	Woodwork	Fall
Fig 9		1 620	54 160	11 ft
Fig 10	(doubled)	1 200	15 400	two of 5½ ft

The puddle mentioned need not be brick clay but any soil that would be

which is amply secured by the plank sheathing

(35) Nature having provided earth and stones close to hand it is desirable that their qualities should be utilised to the utmost with a saving of the perishable wood When permanent works of masonry are substituted for these frail and temporary structures of soft wood and tar paper the stone used in the filling and revetted slopes will be the only material remaining to be again made use of Engineers accustomed to work in wood do not take kindly to masonry structures This was particularly noticeable in Burma where the old wood and mud Engineers of the lower Province before the expansion took place had a holy horror of bricks and mortar the reason is not far to seek Timber structures are quickly and easily put up and require little supervision whereas in masonry and concrete work the arrangements for and collection of mortar stone brick and cement give a great deal of trouble and require careful and thorough supervision the responsibility therefore is much greater

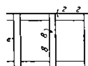
In the States the idea has gained ground that if a structure be of rein forced concrete it must be suitable in point of cost as well as efficiency This is by no means the case as has been proved in several instances taken up in



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Scale of Feet

10 15 2



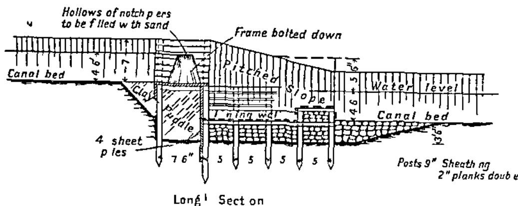
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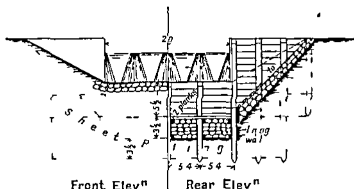
Fig. 92 —Combined Fall and Rapid Calgary Canal

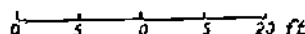
this work To copy wooden structures in reinforced concrete will generally be found on analysis to be very poor engineering

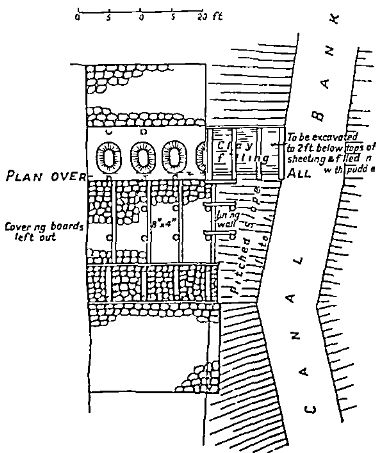
The more modern examples of the great irrigation works in India and

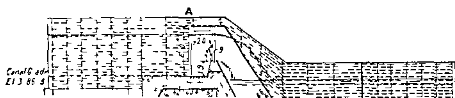


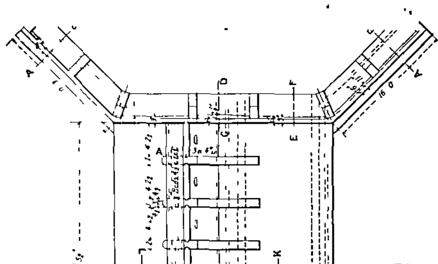
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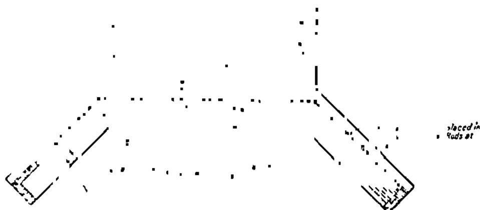
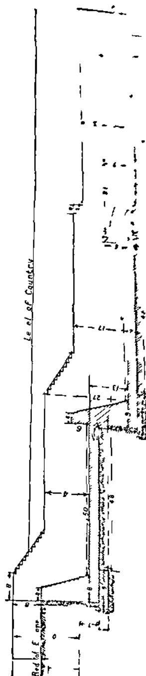


Fig. 11 — Reinforced Concrete Fa

Egypt afford excellent models of suitable construction, not, however, to be blindly copied, but followed with a critical eye to possible defects, and subject to the modification rendered necessary in a country where labour conditions are so vastly different

(36) Through the courtesy of Mr J S Dennis the able head of the Irrigation Administration at Calgary, we are enabled to produce (in Fig 11) the plan in reinforced concrete of a combined regulator and fall which is on secondary canal *C* (*vide* map) in Chap XIII, Fig 2 The drop is 5½ feet The fall is divided up into 7 bays of 4 feet by piers, which continue over the inclined drop wall on to the lower floor

It is presumed that the head to which this work is liable is that acting when 6 feet of water is upheld by baulks let into the pier grooves the channel below being empty If it were not for this fact, the continuation of the piers so as to form buttresses to the thin drop wall would not be necessary With the provision of the rear sheet piling and an impervious upper floor, the drop wall practically consists of the whole mass of enclosed earth and concrete between the sheet piling and its face which thickness is much in excess of requirement Under these circumstances if the case were that of an ordinary fall without regulation, the cheapest construction would undoubtedly be to make the actual drop wall thicker and to do away altogether with the rear sheet piling and the rear floor, which then become superfluous



If on the other hand the existing arrangement of the buttress piers is maintained, the whole of the floor in rear of the pier noses marked 1 together with the sheet piling, is clearly superfluous The upper wing walls would then take off at the end of the shortened abutments *i e* at 1 The drop wall itself would not require any thickening beyond its present dimension of

18 inches In ordinary cases, no upper floor is provided in rear of the weir wall beyond some riprap, and there is no reason why this arrangement



Fig 13—View of a "Wingless" Fall of 14 feet (Burma)

should be departed from unless the foundation is pure sand, which is not the case here.

(37) A fall practically consists solely of the drop wall and the floor, the *side wings not being an absolutely essential feature, as they can be left out,* pitched slopes being substituted The drop wall is simply a lining to the

step in the earthen bed a retaining wall in fact and if built up against the original clay or connected therewith with a puddle backing it should not be subject to hydrostatic pressure Further its weight pressing on the earthen foundation forms a watertight connection below The floor in like manner forms a watertight connection with the earth in the lower bed and with the drop wall Consequently except when the soil is porous as sand lines of sheet piling in rear of the drop wall or below the foundation of the lower floor are not at all necessary and in fact never are employed in any Indian works of this character

A good design in a light material as timber may be quite unsuitable when a heavier material is employed The plan under consideration adheres it is contended too closely to precedents of timber construction It should be borne in mind that in hydraulic works sheer weight is always a desideratum

In reinforced concrete work weight is reduced as much as possible to save expense and it will often be found that it is cheaper to employ the plain concrete of larger dimensions than reinforced concrete

It is considered that this floor should be made at least 3 feet thick the lower half being of closely set riprap covered with 18 inches of plain cement concrete the length also to be increased by half as much again by heavy riprap the wings remaining the same

The remarks regarding the superfluity of the sheet piling do not apply to that underneath the up stream wings In most falls the foundation of these wings is carried down to the lower foundation level and this can be most economically effected by sheeting piles forming the lower 5 feet This line of sheeting should be carried under the abutments and connect up with the drop wall curtain The drop wall might be made vertical without disadvantage with a battered face

The piling shown at the termination of the down stream floor is a useful though not an indispensable adjunct provided this floor were lengthened in concrete to about the same extent as the upper floor is recommended to be shortened The suggested alterations amount in fact to a redistribution of much the same material

The panel counterfort retaining walls are an excellent feature and effect considerable economy in section It is here that reinforced concrete can it is considered be most suitably employed not in the drop wall or floor

(38) A section of a canal escape fall ladder without notches which is worthy of attention is that of the Kushak Falls Agra Canal given in Fig 13 The discharge of the escape its bed width and other essential particulars are as is so often the case entirely wanting Fig 13 is a view of a Wing less Fall

EDITOR'S NOTES

(P) *Notch Falls* (par (-)) —The Notch Fall was invented by Punjab irrigation engineers and after experimental investigations by Mr (now Sir John Benton and others was first brought into use extensively during the construction of the Sirhind Canal system in the period 1872–1882 The

author has taken his diagrams and description of the method of design of these Notches from R B Buckley s excellent treatise The Irrigation Works of India and Egypt which however represents an obsolete type of design in this particular instance The orthodox method of design is set forth in Punjab Irrigation Technical Paper No 2 wherein the standard shape of notch for canal falls is shown as in Figs 14a 14b here inserted It will be noticed that in these diagrams the up stream sides of the notch (in plan) are inclined at 45 degrees to the axis of the canal and the down

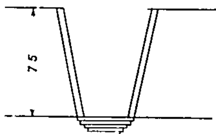


Fig 14a

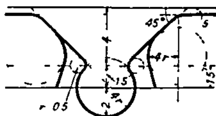


Fig 14b

stream sides at 22 degrees thereby giving the notch a more efficient shape of adjutage for discharge than in the type taken from Buckley in which the up stream angle is about 62 degrees with an angular edge to the mouth piece which is liable to retard flow

The notch wall in Fig 14 is 5 feet thick for a height of 7 5 feet a ratio of 2 3 which is as it should be for stability against water pressure In view of the dynamic action of the current passing the wall on either side a lesser ratio of base to height is not permissible The following is quoted from the Punjab Technical Paper No 2 —

In Mr Benton s experiments it was found that the value of the coefficient of discharge varied from 0 662 to 0 676 the mean value being 0 67 and this has been used in all notch calculations the velocity of approach being also taken into account Observations on distributary notches constructed in accordance with Mr Benton s formula have shown that by substituting 0 70 for 0 67 the velocity of approach may be neglected and considerable labour of computation avoided Experiments on canal notches have not been very extensive but they indicate that by employing a coefficient of 0 78 and neglecting velocity of approach a result is obtained practically identical with that of the more laborious formula Accordingly in the formulæ now given the terms involving velocity of approach have been entirely omitted and these coefficients substituted for Mr Benton s coefficient of 0 67

It is customary for branch canals or distributaries to offtake from their parent channel at points a few score or a few hundred feet up stream of masonry falls and in such cases the discharge passing over the fall to the canal below may be considerably less than that of the canal up stream of the offtakes The width and depth of the canal below the fall may be appreciably different from the dimensions up stream of it yet the notches must be designed to hold up the water to the normal up stream depths whilst passing the *down stream discharges* The details complicate greatly the formulæ of design set forth in the Technical Paper above mentioned but we content ourselves with dealing with the simple case discussed by our

author, of a canal with a full supply depth of 8 feet of water, discharging 2,500 cusecs, both up stream and down stream. The notches have to be designed to discharge correctly at two depths, (d_1) and (d_2), such that if (d_m) be full supply depth and (d_0) be lowest working depth, then $(d_1) = (d_0) + \frac{1}{4}(d_m - d_0)$, and $(d_2) = (d_0) + \frac{3}{4}(d_m - d_0)$.

If we assume $(d_m) = 8$ feet, and $(d_0) = 4$ feet, then $(d_1) = 5$ feet and $(d_2) = 7$ feet. In the case considered we have —

Discharge for depth $(d_1 = 5) = 1{,}136$ cusecs $= D_1$,
Discharge for depth $(d_2 = 7) = 2{,}000$ cusecs $= D_2$.
Other notations are —
l = length of cill of notch in feet
$n = 2 \tan a$, where (a) = inclination to vertical of the sides of the notch
$l + nd_m$ = top-width of notch depth $d_m = 8$ feet
c = coefficient of discharge (0.78 for canals, 0.70 for distributaries)
e = depth to which cill of notch is submerged by tail water
f = drop of fall in feet

Canal Notches with 'Free' Fall—

General equation $D = \frac{2}{3}c\sqrt{2g}(ld^{\frac{3}{2}} + \frac{2}{5}nd^{\frac{5}{2}}) = 5.35cd^{\frac{3}{2}}(l + 0.4nd)$

Whence we deduce

$$n = \frac{D_2 d_1^{\frac{3}{2}} - D_1 d_2^{\frac{3}{2}}}{2.14cd_2^{\frac{3}{2}}d_1^{\frac{3}{2}}(d_2 - d_1)},\quad l = \frac{D_1}{5.35cd_1} - 0.4nd_1$$

Applying this to the data of the case considered, we get
$n = 1.92$ feet, $l = 19.5$ feet, top width $= l + nd = (19.5 + 15.4) = 34.9$ feet.
Full area of waterway $= (19.5 + 34.9) \times 8/2 = 217.6$ square feet.

If we decide to have eight notches, we have for each —

$$\text{Bottom width} = \frac{19.5}{8} = 2.44 \text{ feet, top width} = \frac{34.8}{8} = 4.36 \text{ feet}$$

The corresponding results arrived at by our author are —

Bottom $= 3.3$, top $= 5.7$, area of waterway for eight notches $= \frac{5.7 + 3.3}{2} \times 8 \times 8 = 288$ square feet.

Whence it appears that his notches are 32 per cent looser than those given by the orthodox calculations. This is due to his having reckoned with a coefficient $c = 0.623$, instead of 0.78.

It is better to have notches *a little too tight* than *a little too loose*, because the error of tightness can be more easily remedied without closing the canal by cutting down a course or two of brickwork from the top of the notch walls, as a temporary measure; besides which it is better to have the canal water level a few inches above rather than below the normal required for command of offtaking channels.

In par. (12) our author considers the case of the same canal where it has a fall of 2 feet only. In this case the fall will be drowned at depth d_1 as well as at d_2, and the orthodox formulæ will be —

$$l\left(d_1 + \frac{e_1}{2}\right) + n\left\{\frac{3e_1^2}{4} + e_1(d_1 - e_1) + 0.4(d_1 - e_1)^2\right\} = \frac{D_1}{5.35c\sqrt{d_1 - e_1}}, \text{ and}$$

$$l\left(d_2 + \frac{e_2}{2}\right) + n\left\{\frac{3e_2^2}{4} + e_2(d_2 - e_2) + 0.4(d_2 - e_2)^2\right\} = \frac{D_2}{5.35c\sqrt{d_2 - e_2}}$$

From these equations we deduce —

$n = 5.13$, $l = 18.6$, top width $= 59.6$, area of waterway $= 312.8$

If we have eight notches, the data for each are —

$$l = \frac{18.6}{8} = 2.33, \text{ top width} = \frac{59.6}{8} = 7.45$$

The corresponding data according to our author are

Bottom width $= 4.76$, top width $= 9.24$

Total area of waterway $= \frac{4.76 + 9.24}{2} \times 8 \times 8 = 448$ square feet, o 43 per cent greater than that of the orthodox calculation

Our author decides first how many notches he will have and then calculates the dimension of each. It is simpler and preferable to calculate the dimensions for a single notch, taking the whole discharge of the canal, and then to decide how many separate notches it will be desirable to have

The above formulæ for "drowned" notches are lengthy, but with a little practice the engineer dealing with special cases, especially those of channels discharging less than 250 cusecs, will find that he can simplify his calculations greatly without incurring serious error. In the case of Distributary notches, instead of the shape shown in Fig 14 it is customary to make the sides of the notch in horizontal section simply semi-circular, as in Fig 12 with diameter equal to the thickness of the wall at bed level. As a general rule we think that it will be found correct enough to calculate the notch for two depths, viz, d_2 = full supply depth and $d_1 = \frac{1}{2}(d_2)$. And we think that the assumed coefficient of discharge should be no higher than 0.75 for a "Free Fall," and graduated up to 0.85 for a "Drowned Fall" in proportion to the degree of "drowning"

CHAPTER X

CANAL REGULATION BRIDGES AND ESCAPE HEADS

(1) In places on a canal where an escape or a branch takes off, a regulating bridge across both works is generally necessary Distributaries very frequently take off above falls and the two works are often combined No object however is gained by placing the headwork of the branch directly at the fall and so amalgamating them into one, a better arrangement is to make the branch take off separately, higher up the canal The regulation of falls by means of a superimposed bridge with shutters or baulks which drop on the weir is hardly practicable where notch falls are concerned, and the system is generally to be deprecated as interfering with the regimen of the canal Skew heads for branches have sometimes an advantage The branch could leave the main canal at an angle, the head being recessed so as to admit of the spans being square to the axis of the branch An alternative is a skew head with the piers run forward to the canal side slope

(2) The principles governing the design of canal regulation bridges are not quite identical with those which affect that of canal heads In the latter the object of the design is to enable the head work, if so required either to completely cut off all ingress of the water from the river into the canal or else to admit only as much as may be desirable

The same functions have to be performed by the canal regulation bridges with this difference, that the maximum head of water dealt with is moderate in depth, seldom exceeding 10 feet at the outside, whereas in a river canal head the flood water may rise to anything up to, say, 30 feet above floor level

(3) A design for a regulation bridge and branch head is given in Fig 1 In this case the water in the canal is assumed to be 8 feet deep and that in the branch 6 feet, the floor of the latter being placed 2 feet above that of the main channel The banks, with cutting and spoil, are assumed to be 15 feet above canal bed level and the bridge roadway or platform is placed at this level The spring of the arches is fixed 2 feet above F S L, which is the same* in the branch as in the canal In the canal regulation bridge the spans are made 10 feet Large spans with high, heavy arches are quite out of place in irrigation canals, where headway need not be provided for laden barges Most of the examples of existing canal regulators given in Mr

* This is not a good arrangement There should always be a difference of water level of at least 2 feet between main canal and branch canal or of at least 1 foot between canal and distributary to allow of a crest wall and silt excluding arrangements between parent channel and off taking channel —Ed

Buckley's "The Irrigation Works of India" are of this latter type, which it would be folly to imitate in a purely irrigation canal The openings in the

Figs 1 1a 1b 1c 1d 1e —Design for Regulator and Branch Head

skew branch head which takes off at an angle of 60 degrees with the axis of the main canal are made 8 feet wide, which will give an oblique span

of about 9 feet The skew arches have the same rise and thickness as in the canal regulator

(4) The disposition of the wing walls in the combined work is arranged so as to allow of a suitable approach to the bridges from the canal bank road on either side, as well as from the country at the rear The connection between the two works is formed by a short continuation of the abutment of the canal regulator, which joins on to the end of the left skew abutment of the branch This is the only vertical retaining wall in the design The width of the roadway between parapets is 12 feet This makes the length of the piers and the floor in both cases about 24 feet The head of water being only 8 feet, the proportional length of the masonry floor comes to $3H = 24$ feet, or measuring from the grooves, over $2H$, so that the width of the masonry floor is if anything, in excess of requirements, and need not be extended beyond the down stream pier noses The thickness of the floor is $\sqrt{H} = \sqrt{8} = 2\,8$ feet $+ \frac{S}{10} = 3\,8$ feet in the main regulator, and $\sqrt{6} + \frac{S}{10} = 2\,5 + \,8 = 3\frac{1}{4}$ feet in the branch, the subsoil being assumed to be firm clay H is the depth of full supply above floor, and S the span The addition of $\frac{S}{10}$ is for the piers, to allow increased depth for distribution of weight, which answers well in practice

(5) The gates will be lifted by a travelling winch, which will run on rails clear of the roadway One rail will rest on the dwarf parapet wall provided for that purpose, and the other will be bolted on to a rolled beam some 9 inches deep built into the top of the up stream cut-waters, which for this purpose are continued up to road level The detail of this arrangement is shown on a larger scale in Figs 1d and 1e The gates, which only require a watertight connection between each other, will be in plan as shown in Fig 1e They will, as a matter of course, be fitted with roller attachments

(6) The arrangement of the pitching below the two works is shown in Figs 1, 1a and 1b That on the canal slopes is taken 2 feet above the water line and terminates in a slope of 45 degrees Where the sloped pitching projects beyond that in the canal bed, a narrow slip of horizontal pitching extends on either side to form a foundation for the toe of the slope

(7) Fig 2 is a presentment of the Raswaniya Regulator on the canal of that name in Lower Egypt With the exception of the down stream wings the alignment of which presents some novelty the design is but a copy of the old Ganges Canal types

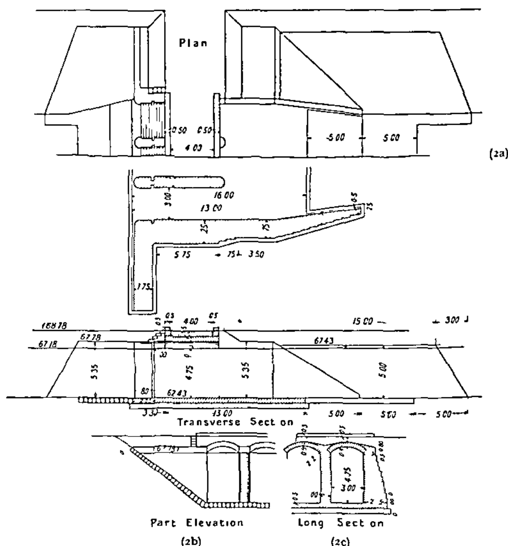
The following points are noted —

(1) Considering the depth of water viz over 15 feet, the spans are very narrow

The width of openings, except in the case of weir sluice heads should

not as a rule be much less than the depth of water, and as the regulation of this work is believed to be only partial spans of 4 or 4½ metres *i e*, 13 to 15 feet could well have been adopted The regulation is effected by the old primitive method of wooden sleepers or baulks dropped into grooves cut in the brick piers

(2) In conformity with the practice in Upper India, the wing walls are



FIGS 2 2a 2b 2c —Raswaniya Regulator (Egypt)

vertical in section which is the least economical form The direct return up stream wings are likewise an unsuitable disposition unless for a head regulator taking off from a river bank which is apparently not the case here Reference to the elevation in Fig 2 will demonstrate how injuriously the sudden narrowing of the water way due to the abrupt entire suppression of the side slopes must affect the current A good arrangement of the water wings will on the other hand ensure as gradual a change as possible in the

sectional area of the water way, and this can only be effected by either adopting splayed walls which gradually absorb the side slopes, or else a direct continuation of the abutments by means of a sloping crested wall, similar to what is adopted down stream in this design, but parallel to the axis of the canal, which type of water wing effects a similar gradual widening, but in a different manner

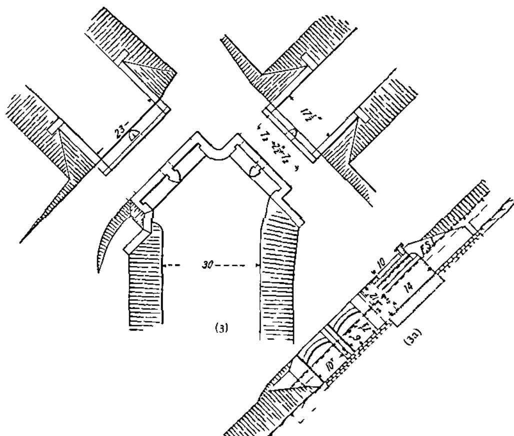
(3) The down stream wings in this work converge on plan, the object being to force the current into the centre of the water way It is believed that this device will defeat the very object which it is intended to secure The contracting of the water way will tend to cause the current, on being released from the restraint of the wings, to spread out on both sides, as water flowing through an adjutage undoubtedly does

A good point in the design is the arrangement of the pitching on the slopes, which is as it should be

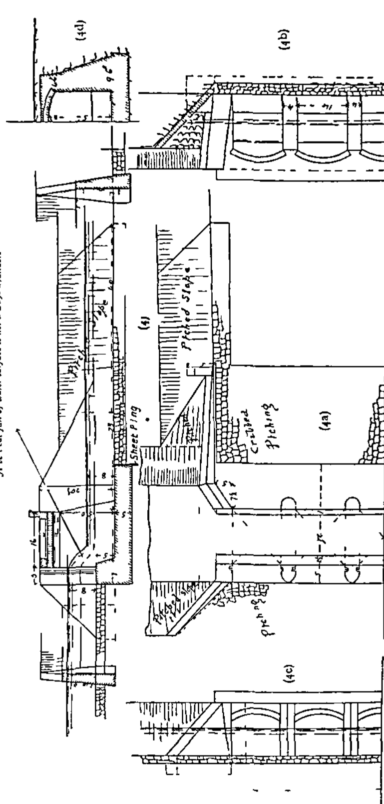
(4) The floor of the work is 1 metre or $3\frac{1}{4}$ feet thick, which is sufficient, having regard for the narrowness of the spans

The length of the floor is, if anything in excess of requirements

(8) Fig 3 is a design of a bifurcation The depth of water is taken as 6 feet in the canal and also in the two branches It does not call for much



Figs 3 3a —Design for Bifurcation



FIGS 4 4a 4b 4c 4d.—Design for Canal Escape Head.

comment, the wing walls have been shown as vertical in order not to complicate the sketch unnecessarily. A design for a regulator without bridge to pass traffic is given in the next section on Canal Escape Heads (Fig 5)

Canal Escape Heads

(9) Notwithstanding the fact that the admission of water at the head of a canal may be under complete control, still emergencies may arise when it may be necessary to shut off, or reduce supply, by reason of a breach of the canal embankments or by reason of local rainfall which may stop the demand for canal water. The canal head may be very far off, say 50 or 100 miles distant, from the site of the emergency, in which case it might take two or three days for a canal head closure or reduction of supply to take effect where needed. For this reason escapes in suitable positions, whereby the surplus supply can be got rid of, are sometimes useful. Their existence, however, is apt to lead to careless regulation of supply, and to waste of water and interruption of irrigation, so their use should be discouraged except on occasions of real necessity.

(10) The position of an escape head should be as close as possible to a drainage line, so as to diminish the length of the excavation in the channel. Falls may have to be constructed along its course, in order to overcome the difference of level between the bed of the canal and that of the river or stream into which the escape channel will tail.

Canal aqueducts crossing drainage lines can sometimes be utilised with advantage to pass off surplus water. An example of such a work is given in Fig 7, to which reference will subsequently be made.

(11) Fig. 4 is a design for a canal escape head combined with a fall under the following assumed conditions —

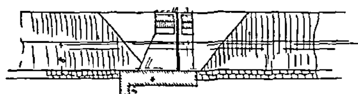
The bed width of the canal is 100 feet, side slopes 1 to 1, the depth of water, full supply, is 8 feet, and mean velocity is $2\frac{1}{2}$ cubic feet per second, having a discharge of 2,160 cubic feet.*

(12) The design of the escape head in Fig 4 consists of a bridged overfall weir, the length being divided into seven bays of 14 feet. As the bridge will be used for traffic, being on the canal bank road, the width will be 12 feet between parapets. The arrangement for the gates will be similar to that in the example given in Fig 1, the outer rail for the traveller being supported by a rolled girder built into the top of the front cut waters. The depth of film (d) is taken as 8 feet (that of full supply in the canal) for purposes of sectional calculation.

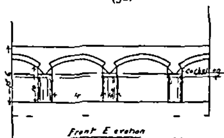
The disposition of the wings down stream (Fig 4a) is of the dog legged

* An escape channel may sometimes be used as a sluiceway for scouring silt from the bed of its parent canal, but this is not the primary function of its design. In order to function in that way it would need an outfall far steeper than that of the canal. Even if such out-

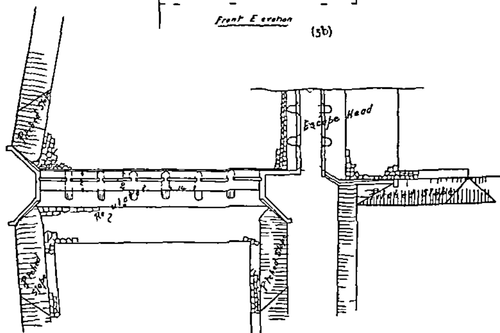
type the up stream right wing is of the splayed sloping crested type to allow easy access of entry for the water in the channel with regard to the left wing down stream (*vide* Fig 5) the junction with the regulator is formed by a continuation of the abutment on a reduced section



(5a)



(5b)



FIGS 5 5a 5b —Design for Regulator below Escape Head

The dimensions of all the parts are in exact accordance with the formulas and rules adopted by the author as detailed below —

Rise of arch 2 feet $= \frac{S}{7}$

Thickness of arch $1\frac{1}{2}$ feet $= 4\sqrt{r} = 4\sqrt{12} = 1\ 4$

Thickness of abutment at springing $= \frac{r}{5} + \frac{V}{10} + 2 = \frac{12}{5} + \frac{2}{10} + 2$

$= 4\ 6$ feet

Thickness of floor $= \sqrt{H} + \sqrt{d} = \sqrt{8} + \sqrt{5} = 5\,0$*
Length of floor beyond weir wall $- 2\,(H + d) = 2 \times 13 = 26$ feet
Thickness of piers $= 3S = 3 \times 14 = 4\,2$ (taken as 4 feet)

The thickness of the base of the retaining walls is in accordance with the formula $4H - 1$ for stone walls with a face batter of 1 in 10 The up stream wing has this proportional thickness at the canal bed level below ground the back is carried down vertical the face continuing to batter to the foundation

(14) Fig 5 is the design for the canal regulation bridge just below the escape head It is not supposed to be required to cross traffic but only for regulation purposes

The spans adopted are the same as in the escape head viz 14 feet of which there are seven giving a combined width of waterway of 98 feet *i e* closely approximating to the bed width of the canal which is quite sufficient for bridges and cross canal works The canal bed is shown widened out on one side only

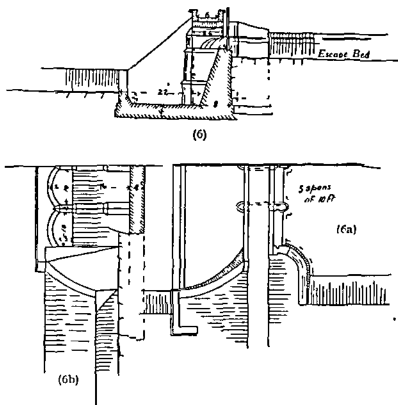
(15) We will now proceed to give an example of an existing escape head with fall which is on the Godaveri Eastern Canal

Figs 6 6a and 6b are drawings of this work The depth of the fall is exceptionally great being 16 feet above floor of fall and 13 feet above the level of the escape channel below The latter being 3 feet higher than the floor a water cushion 3 feet deep is formed by means of a raised curtain wall The weir is divided by piers into five spans of 10 feet The piers are partly built over the weir and the roadway is only 6 feet wide between parapets From this it is evident that the bridge is not required for traffic but is merely a foot regulation bridge This work seems on the whole to be well planned although the curved wings are not commendable Straight battered wings would answer the purpose just as well and be easier to build besides which the railway under bridge style of curved wings adopted down stream which is so common in all Madras works is objectionable their curved shape tending to encourage a rotary movement in the water below the weir termed pooling which is very destructive in its action

The piers are economically designed the thickness is $3S$ at the base and they taper by insets to 2 feet or $2S$ at the springing of the arch of the bridge Piers in a fall are subjected to very slight water pressure end on and receive great support from the weir wall into which they are built They are not likely ever to be subjected to lateral pressure as in the case of a regulating head or bridge consequently the thickness can safely be diminished to that suitable for an ordinary bridge The depth of water liable to pass over the weir is probably not more than 3 feet or 4 feet judging from the height of the banks of the approach channel The closure is effected by wooden gates raised by screw gear A small movable winch on

* But this feature must be governed by the nature of the foundation soil —Ed

wheels which could be taken away and housed when not required, would probably form a better lifting apparatus, or else a row of simple wooden windlasses fixed on the cut-waters Screw gear is suitable for reservoir sluice gates in deep water, which are moved 2 feet or 3 feet up and down at



FIGS 6 6a 6b —Escape and Fall on the Godaveri Eastern Canal (Madras).

the outside, but in the Madras Irrigation Department it is applied to every kind of water gate from undersluices to waste weirs

(16) The floor of this work is only 4 feet in thickness The nature of the foundation soil is not known, but as the work has existed for over half a century, the floor is presumably strong enough

The cistern forming a water cushion 3 feet deep, is in accordance with Dyas' formula, as given in Molesworth's Pocket Book (see Chap IX, par (26))

The weir wall is 4 feet thick at top and 8 feet at the bottom Being 16 feet high, from crest to floor, it would need a base thickness of $\frac{16}{3} = 5.33$ for earth pressure, or of $\frac{16 \times 2}{2} = 10.66$ for water pressure The thickness adopted $\frac{16}{2} = 8$ feet, seems suitable, and has proved so.

(17) It has already been noted that escapes for surplus water can easily be arranged for at aqueducts or drainage crossings the escape or surplus weir being combined with the latter work An excellent example of such combination is given (Fig 7) of a work on the Connamur Canal Madras

The canal here crosses a drainage line which is taken underneath it by a syphon or syphon aqueduct This consists of two culverts 8 feet wide and about the same height The waste weir and regulating and traffic bridges are built at the end of these two culverts half the weir being simply a dwarf wall built over the end of the barrels the other half lying beyond them The three centre piers are spaced 9 feet apart and are built in part directly over the central pier and the two abutments of the syphon The weir con sists however of four spans of 9 feet two of which lie outside the syphon altogether and here the weir wall is of the full depth of 14 feet Both abutments are therefore quite clear of the syphon

(18) An example of a very large escape which belongs more properly to reservoirs than to canal works is the Koshesha Escape in lower Egypt

Figs 8 are plans of this immense work which consists of sixty spans of 3 metres or 10 feet and it upholds 6 2 metres or say 22 feet of water although it is stated in Egyptian Irrigation to be subjected to a head of 4 metres only This is too vague to enable a calculation to be made of the horizontal thrust of the water on the work though it is sufficient for the vertical pressure on which the design of the floor may be made to depend The soil is good hard clay

The main peculiarity of the design is the division of the waterway into two parts by arches thrown between the piers which thus form an inter mediate platform The upper and larger portion of the vent above this platform is closed by iron gates hinged at the bottom which when released fall automatically on to the platform to which they are hinged They cannot be raised when the level of the water is much above that of the platform but are intended when the reservoir has to be emptied to remain down till the water has run off no regulation by them being possible The vents below the platform are closed by ordinary draw gates working in grooves these only come into use when the water above the level of the platform has been drained off A 12 feet roadway between parapets is pro vided so that evidently provision has to be made for crossing traffic This and the considerable projection of the piers in front of the gates apparently to allow a wide space between an outer set of emergency grooves and the face of the barrier gates causes the piers to be of exceptional length which is not required for statical resistance This work is evidently erected in the middle of a long embankment crossing flat country similar to a sluice in a tank embankment it is not situated in a depression or marked natural watercourse

The arrangement of longitudinal rubble spurs above and below the escape to guide the current in a direction normal to the axis of the work is admirable and so is the disposition of the abutment wings

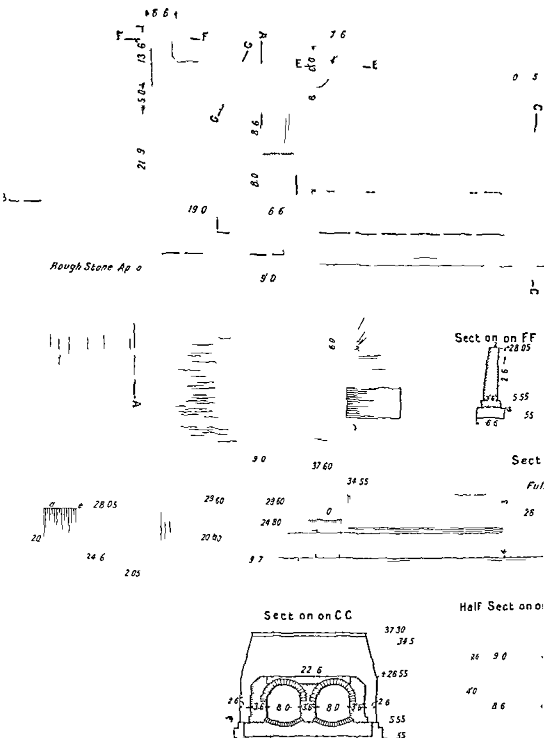


FIG 7—Comb ned Syphon and Escap

(19) In some other points the design of this work is, however, open to criticism

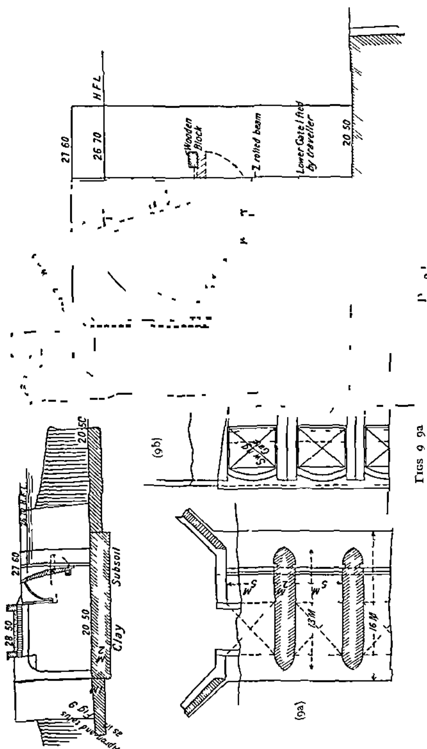
Taking the floor first, the depth under the piers, which depth runs right through, is considerable, being 2 75 metres, or 9 feet Considering that the soil is hard clay, and the head of water nothing very remarkable, being 7 metres, or 22 feet, when the bed is below dry, but stated not to exceed 4 metres, or 13 feet, this mass of rubble masonry appears excessive

The Assiût works built not on hard clay but Nile mud, have a depth of foundation of 3 metres, or 10 feet, with not far short of the same head, viz , 3 25 metres in the Ibramiya Head

With regard to the superstructure The provision of spare grooves and the consequent extension of the piers up stream, a construction always to be avoided if possible, appears an unnecessary precaution, particularly in view of the fact that the work will only be used once a year, and will doubtless remain clear of water for several months, when any repairs to gates, etc , could be carried out The floor should, however, be so constructed that it will never require repairs , this can always be ensured by using good cement masonry of sufficient depth

(20) The division of the spans horizontally, though doubtless an excellent arrangement in itself, it carried out by the insertion of a great thick vaulted masonry platform, which takes up a great deal of the waterway, necessitating more spans to carry off the discharge than would otherwise be requisite This obstruction is not short of 5 feet in depth Here is a case in which iron could be used with great advantage in place of masonry The falling gate, hinged at the bottom, necessitates a long platform to receive it when down If a gate hinged at the centre of pressure, *i e* , at one-third of the height, or thereabouts, be substituted it could be manipulated with great ease, even under full pressure, that on the upper and lower leaves being balanced It could also be arranged to fall automatically when the impounded water rises above a fixed level A balance gate of this description is hinged at both ends on a steel axle, which is either built into the piers or else is received into a movable plummer block, which, provided with anti-friction rollers, can slide up and down an iron groove The gate, after being pulled into a horizontal position, can then be raised by chains attached to the blocks in the grooves when offering the least resistance to the water pressure This, in certain cases is a more suitable arrangement than that adopted in Fig 9 (*post*) The total pressure on each axle will be $\frac{wlh^2}{4}$ or, in the case shown in Fig 9c, $l = 16$ feet, $h = 13$ feet and $w = \frac{1}{36}$ ton, whence the expression becomes $\frac{16 \times 169}{4 \times 36} = 19$ tons, or taking 6 tons as safe shearing stress for steel, the sectional area of the axle will be $\frac{19}{6} = 3\frac{1}{8}$ square inches nearly, requiring to be 2 inches in diameter

(21) Considering the depth of water (viz , 22 feet), the spans of 10 feet width are decidedly too narrow They could be well increased to 5 metres,



Figs 9 9a

or 16¼ feet, the pier being 2 metres wide or 4S A revised design comprising these alterations is given in Fig 9

The spans are increased to 5 metres and in place of the heavy masonry

platform, a Z-shaped rolled beam is substituted, on the upper flange of which the base of the swing gate abuts, while the lower draw gates presses on its lower flange The upper swing gate is 4 metres, or 13 feet, the lower draw gates $2\frac{1}{2}$ metres, or $8\frac{1}{4}$ feet deep The latter can be raised $3\frac{1}{2}$ feet while the upper is down The depth of the free waterway is greatly increased by this arrangement, viz, from 5 metres to 6 metres, or from $16\frac{1}{2}$ to 20 feet The sixty spans of 3 metres give an effective waterway of $60 \times 3 \times 5 = 900$ square metres, or say 10,000 square feet With the alterations shown in Fig 2, each span of 5 metres has a free waterway of $5 \times 6 = 30$ square metres, the required number of spans of 5 metres will then be $\frac{900}{30} = 30$ spans, giving a length between abutments of $(30 \times 5) + (29 \times 2) = 208$ metres The existing work has a length between abutments of $(60 \times 3) + (59 \times 1\ 3) = 257$ metres, the revised design will thus cost 20 per cent less, and with the further reductions in the superstructure and floor (if the latter were reduced to reasonable dimensions), the saving would be quite 25 per cent

(22) A rough sketch of the swing gate, an invention of the author, is given in Figs 9 and 9c It is shown clear of the archway, but could be fixed underneath it if so desired

The arrangement for revolving the gate consists of two endless chains attached to projections on either side of the top of the gate, which chains work inside curved iron grooves Each chain at the top of the piers is carried round a winch drum connected by a horizontal shaft revolving which either way causes the gate to be lowered or raised to any extent If the gate is required to fall automatically, or at once when released in the former case the axle can be fixed at such a height that when the water reaches a certain point the gate will revolve of its own accord overcoming the friction of the chain and winches If, on the other hand, the gates are required to be suddenly released with full supply on, the winch drums can be prevented from turning by a catch, which being knocked away the gate will be free to revolve

In the design in Figs 9b and 9c the gate is not intended to be lifted, but when horizontal to remain in that position with water passing above and below it If it is deemed desirable to lift the gate grooves can be provided as shown dotted with blocks for containing the axles of the gate These blocks are attached to chains lying in the vertical grooves by which the whole gate can be lifted up by a traveller The arrangement adopted in the plan of pivoting the gate on a fixed axle on which it swings and leaving it *in situ* is simpler and better, the only objection to this arrangement being the possible accumulation of floating *debris* on the end of the gate This, however, can be easily got rid of by turning the gate round a little for a time when it will be swept away by the current

(24) The design of distributary heads should be similar to that of canal branch heads or cross canal regulators *i e* an open waterway with or without a raised sill, closed by draw gates

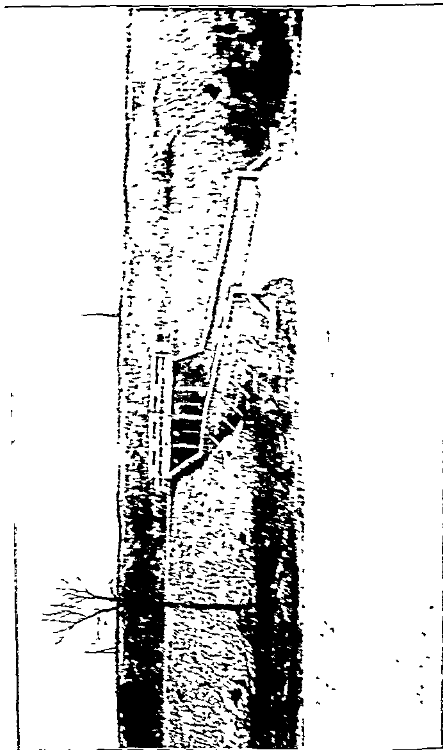


FIG. 10.—View of Escape Head on Calgary Canal, Alberta, Canada.

For such works double wooden gates can well be used. These can easily be fitted with small iron rollers, the lateral staunching being effected by vertical staunching strips fastened to the gate at their upper extremity only.

The pressure of the water forces these against either the base or the side corners of the tables of the iron grooves These gates can be easily manipulated by a wooden windlass fixed over each span, the barrel fitted with ratchet and pawl The windlass can be turned by detachable wooden bars fitted into holes, or else a spoke wheel can be fixed in the centre of the drum. The system so much in vogue in Madras, of constructing canal and distributary heads on the principle of small sluice ways situated at floor level, which are closed by a gate operated by screw gear, is deprecated

(25) A view of an escape head, situated a mile or so below the intake of the Calgary Canal, in Alberta, Canada, is given in Fig 10

The structure, as will be seen, is of wood and consists apparently of 5 vents, each 5 or 6 feet in width, closed by draw gates, which are operated by rack and pinion hand gear It forms a very instructive example as to the use to which timber can successfully be adapted This work, together with all other timber erections on this canal, will, however, eventually have to be replaced by permanent structures in masonry or concrete

Possibly in some future day these arid canal banks will be planted with umbrageous trees, and thus come to bear some resemblance to the shady avenues on the older Indian canals which in a thirsty land add such a charm to the landscape

(26) Fig 11 is a view of a bifurcation on the same canal The regulators are fitted with the rear pivoted balance gates illustrated and described in

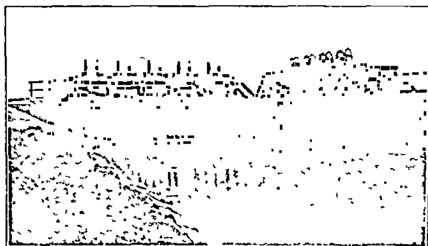


FIG 11 Bifurcation on Calgary Canal

Chap VIII This arrangement though doubtless effective and easy of manipulation has, it must be confessed an extremely crude appearance These temporary structures are it may be observed by no means inexpensive It is believed that a properly designed masonry work would cost very little if any more

The policy of the Canadian Pacific Railway in this canal, as in their own particular case was, however, to get the work *through* somehow as soon as possible, and then subsequently improve gradually when leisure and opportunity offered Who will say that in a pioneer project of this kind this was not the proper policy to adopt ?

CHAPTER XI

CANAL CROSS DRAINAGE WORKS

(1) THE disposal of drainage that is intercepted by a canal can be classified as the following —

I By lateral diversion *i e* by excavating a channel parallel to the canal the stream can be thrown into another drainage line for the disposal of which provision has been made

II By passing it underneath the canal the canal either crossing the *stream on a raised aqueduct or if the headway is insufficient* for a clear passage the bed of the stream is depressed below normal level and the water passes in a tunnel underneath rising again on the further side This latter is termed a syphon or syphon aqueduct

III The drainage water can be admitted into the canal itself This is termed an inlet

IV The drainage can be taken into and across the canal at the level of the bed of the latter the inlet on one side and the exit on the opposite side when both channels are at approximately the same level This involves one regulator across the canal and one at the further bank across the exit of the drainage This is termed a level crossing (Figs 18 and 19)

V The drainage can be taken over the canal by an aqueduct This is termed a superpassage to distinguish it from an aqueduct passing a canal over a natural stream or drainage line

VI The canal can be taken under the drainage line by a depressed syphon

(2) In Fig 1 we have an example of Class I being the Thori Nali Aqueduct on the Midnapur Canal In this work the canal bed is 25 feet above that of the river giving sufficient headway to pass the highest flood which is 18 feet deep

Like all masonry aqueducts the construction mainly consists of an arched bridge with platform at canal bed level and provided with two solid parapets which retain the water flowing through

To reduce expense the waterway of an aqueduct is often made narrower than the average width of the canal in earthen banks Owing to the smoothness of the sides the coefficient of rugosity (n) is much less than that applicable to channels with earthen sides and bed being 013 in the former against 025 or 0225 in the latter This alone greatly increases the velocity so that a considerable reduction in section can be effected even if the original mean velocity of the current were retained As however that velocity can safely be increased in the masonry channel in cases where conditions permit a still further reduction in the width of the waterway in the aqueduct can be

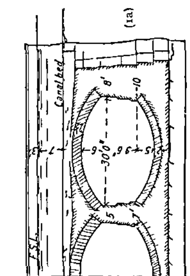
effected In the example we are considering, the bed width is apparently not reduced (Fig 1d)

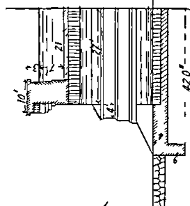
(3) The thickness of the arch throughout is $2\frac{1}{2}$ feet, the curvature being 90 degrees The value given to n in the formula $n\sqrt{r}$, adopted for crown thickness (*vide* par (17), Chap IV), is therefore 55 The weight of water overlaying the arch is $7 \times \frac{1}{35}$ tons $=$ ·2 ton per square foot.

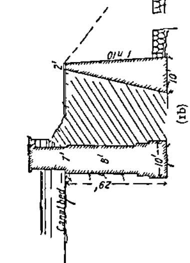
The live load in ordinary bridges is assumed at about 120 lbs per square foot This represents an equivalent weight of water of 2 feet in depth Therefore the extra live load, which, however, is not suddenly applied and removed, as in the case of railway and road bridges, will be the weight of the depth of water carried, minus 2

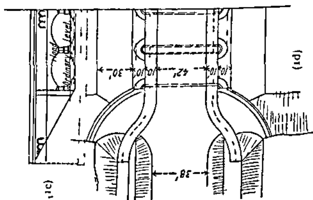
The value of the coefficient n in the formula $n\sqrt{r}$ can then be approximately increased in proportion to the depth of water carried, in ordinary cases $n = {\cdot}4$

The following rule for deducing the increase to the value of n will suit in most cases Let d = depth of water, then $n = 4 + 02\ (d - 2)$ When the depth of water is 2 feet, n will remain 4, with 5 feet carried $n = 46$, with 6 feet $n = 48$, with 7 feet $n = 50$, with 10 feet $n = 56$ This is for large spans of









FIGS 1 1a 1b 1c 1d —Thora Na'a Aqueduct, Midnapur Canal

over 25 feet For smaller spans n should be taken as 5, no matter what the depth of water is

In the example of the Thora Nala Aqueduct the crown thickness would then be, with 7 feet water carried, $5 \times \sqrt{r}$ or $5 \times 4\ 6 = 2\ 3$ feet

The parapets, as is usual in large aqueducts, are widened out to carry a roadway, as communication for cart traffic must be kept up along canal banks The parapets here are 7 feet wide at base and 6 feet at top, corbelled out to provide a 10 feet roadway, with an iron rail fence on either side The thickness of the parapet in this case is excessive To resist water pressure it need not exceed two thirds the depth of water

(4) The piers are $\frac{S}{6}$ or $167S$ in thickness They widen out by offsets to 7 feet, or $23S$, at the base, S being the span

There is no definite rule regarding the ratio of the thickness of piers proportional to the span in the case of large span bridges It may be taken to vary from $\frac{S}{10}$ to $\frac{S}{5}$

For heavy works of this description the proportion $\frac{S}{6}$, as in this case would not be excessive *In the Kali Nadi Aqueduct of 60 feet spans (Fig 3)* the proportion is $\frac{1}{8\frac{1}{2}}$, and in the Gunneram Aqueduct (Fig 4), with spans of 40 feet, the proportion is $\frac{1}{7\frac{1}{2}}$ nearly or, to be exact, $15S$ In the Budhi Superpassage (Fig 10) the proportion is $\frac{1}{7}$ All these carry about 7 feet of water, and are all built of brickwork *

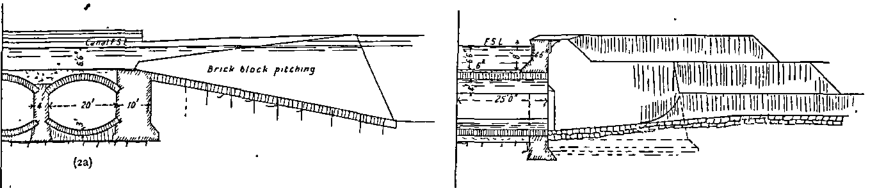
The piers of large span bridges should increase in thickness towards the base with the object of better distributing the pressure on the foundations, and further of inducing a uniform stress at all points in the height of the pier A formula for effecting this is given in "Molesworth s Pocket Book," p 89

In the example we are now examining the width of the pier increases by two offsets from 5 feet to 7 feet at the base A straight or curved batter would have been a simpler and better construction

(5) The floor is composed of inverted arches with a versed sine of 5 feet, the thickness in the centre of the span being 4 feet, and that at the spring line of the inverts 9 feet The object of this invert is evidently to distribute the weight on the piers evenly over the somewhat shallow foundation †

* A suitable rule to adopt would be to make the top thickness of brickwork piers of aqueducts sufficient to leave a thickness of at least 1 foot between haunches of arches at their extrados say $(t) = 1 + 0\ 8\sqrt{\text{radius}}$ —Ed

† It is a mistake to design an invert with a flatter or smaller angle of curvature than that of the overhead arch for that means a greater radius and greater horizontal thrust which tends to displace the abutments Yet it is the usual practice to design inverts in the manner here condemned with the result that many engineers regard inverts generally with suspicion The curvature of an invert should not be less than a quadrant as that is the curvature of least horizontal thrust but in any case it must have some tendency to lateral displacement of abutments unless its horizontal thrust be balanced by some outside force —Ed



FIGS. 2, 2a, 2b, 2c, 2d —Kerai Aqueduct, Sôn Canal

(6) The sections of the abutment and of the wing walls (Fig 1b) are good Figs 1c and 1d are part elevation and plan at a smaller scale Reference to Fig 1d will show the disposition of the wings In almost all aqueducts and superpassages double sets of wings are required viz two long curved land wings to form the connection between the masonry aqueduct and the earthen banks of the approach channel and two water wings connecting the face of the abutments with the river banks on either side The land wings form really a continuation of the parapet walls and are of the same section at the top Being subjected to hardly any earth pressure they can be built with vertical sides of the same width throughout as the top

(7) Figs 2 2a 2b 2c and 2d are different views of the Kerai Aqueduct on the Son Canal The construction is very similar to the last example the main peculiarity of this design being this that to obtain the necessary headway the spring lines of the invert arches are made on a level with the bed of the drainage line the crown and floor proper being depressed 3 feet below the normal bed A down stream pitched slope connects the two levels If the flood line rises above the archway, the aqueduct will become a syphon aqueduct The drawing in The Irrigation Works of India from which this figure is prepared does not however show the flood line This device of depressing the floor to avoid the obstruction offered by the invert is a good one *i e* on the assumption that the invert is necessary The bases of the piers are carried through the concrete to the ground—a bad arrangement which tends to concentrate instead of distributing the weight of the super structure The thickness of this mass floor is determined by the limit pressure allowable on the soil The bearing area of the pier base increases 1 foot in width for every foot in depth of the floor (par (9) Chap IV)

The invert stops dead short at the termination of the piers the sloping continuation peculiar to the last example not being adopted

(8) The parapet in this example is $4\frac{1}{2}$ feet thick or just about $\frac{3}{8}D$ it is widened inwards at junction with the arch by the addition of a triangular strip thickening the actual base to 8 feet and filling up the corner This is an excellent arrangement The widened roadway along the top of the parapet is formed by a projecting cornice on the inner side and by iron brackets probably planked over with a fence on the outside This forms a roadway 8 feet wide giving just room for a cart to pass The arch is 2 feet thick *i e* very nearly $54\sqrt{r}$ the radius being 14 feet

The section of the abutment which is 10 feet thick or half the span and with the buttress (*vide* Fig 2b) is 16 feet thick in the centre may have been designed specially heavy in order to resist thrust of inverts The bed of the canal is apparently not made up level with the top of the abutment but slopes downwards towards the ground surface This slope is pitched and likewise both the inner slopes of the canal banks

The disposition of the wings is shown in Figs 2b and 2c The water wings are semicircular in plan and are very high so that the land wings are subjected to very slight unbalanced earth pressure The widening of the

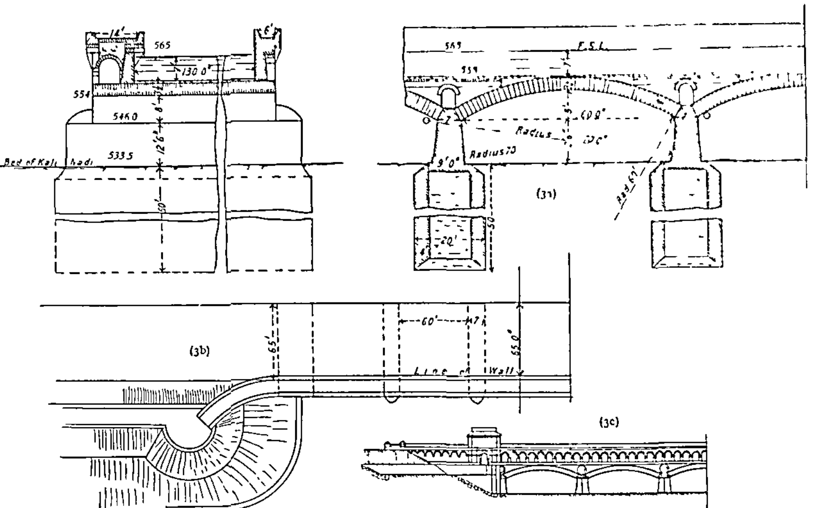
canal from 40 feet to 120 feet is shown in Fig. [illegible]c. The semicircular water wings form a protection against the scour of the river in flood.

(9) Fig. 3 is a representation of the Nadrai Aqueduct over the Kali Nadi on the Lower Ganges Canal. This is one of the largest single works erected in Upper India, consisting of fifteen spans of 60 feet and a width between parapets of 130 feet. This is the second work erected at this site within a short period. The first aqueduct * was designed to pass a maximum discharge of 18,000 cubic feet per second in the stream, and was completed and in working order when, after a period of exceptionally heavy rainfall, the Kali Nadi, which is a natural drainage depression, the banks and bed being cultivated, came down in an unprecedented flood. With the exception of one railway bridge, which was founded on deep sunk iron caissons, every single bridge on this long watercourse was destroyed; the aqueduct also was washed clean away and disappeared completely from sight, being buried in an immense hole scoured out owing to the obstruction. The loss of this aqueduct was a most serious disaster, as, being situated not far from the head work, the irrigation of the greater part of the canal was completely closed, involving great loss of revenue. A new aqueduct near the old site was immediately put in hand, the design being entirely recast to provide for passing 140,000 cubic feet per second. The new design consists of fifteen spans of 60 feet, the piers, abutments and wings being founded on circular wells 20 feet in diameter, sunk 50 feet below the bed of the stream. This structure is not provided with a floor, owing to the great depth of the pier foundations, which are regarded as being below the influence of any scour.

(10) The area of clear waterway in each span is 1,140 feet, and there are fifteen spans—total 17,100 square feet. This area has to pass 140,000 cubic feet, hence the velocity of current passing the bridge will be $\frac{140,000}{17,100} = 8\frac{1}{4}$ feet per second. This immense velocity would speedily scour out the bed till equilibrium was produced. In the original project it was proposed to place a floor 10 feet below the normal bed level of the stream to allow for scour down to this level. This would enlarge the area of the waterway to 24,000 square feet and decrease the velocity to 6 feet per second. This velocity again is excessive, and if the floor as proposed were actually constructed further scour might still occur in the unprotected river bed immediately below. This hypothesis is borne out by the recorded action of subsequent floods, one of which is stated to have scoured out a hole 30 feet deep † below the work. From the above it is evident that the depth of pier foundations designed, viz. 50 feet, is in no way in excess of requirements. The afflux is estimated to be 6 inches.

(11) With regard to the superstructure, as the flood rises as high ‡ as the bed level of the canal, the arch and spandrels of the bridge openings cause

* See Editor's Note at the end of this Chapter.—Ed.
† [illegible] is a stratum of clay at this depth in the subsoil.—Ed.
‡ It rose to 56.5 or 8.5 feet above crown of arches down stream of of July 1[illegible]th, 1885.—Ed.



FIGS. 3, 3a, 3b, 3c—Kali Nadi Aqueduct, Lower Ganges Canal. Fifteen Spans of 60 feet

considerable obstruction to the waterway. This obstruction, exclusive of the piers, is no less than 400 square feet in a water area of 1,600 feet, *i.e.*, one-quarter of the area of the waterway above the normal bed level of the river. The obstruction due to the pier cut waters is not included in this estimate. This constriction of the passage is unavoidable in the arched type of superstructure in all cases where the flood level is higher than the crown of the arch, but the adoption of a steel girder superstructure for the aqueduct would very largely reduce the obstruction. In that case the only obstruction below the water line would be the joists and longitudinals below the plate floor, these would not take up more than 2 feet in depth, and the proportionate obstruction would be only one twelfth of the normal waterway. However, under the peculiar circumstances of this particular case the relief thus afforded would be comparatively small. The congestion occurs mainly in the base of the water section, which is relieved by scour of the bed at each flood. Consequently the necessity of deep isolated foundations to the piers would still remain, as also the inadvisability of attempting to curb the bed scour by the imposition of a floor, even if sunk below the normal bed.

(12) In rivers, whose bed is composed of micaceous sand of low specific gravity, as the Ganges, Jumna, and the Kali Nadi in question, the bed level *is being constantly altered by flood action, the water section automatically* adjusting itself to varying discharge and velocity. During the prevalence of high floods the sandy bed is scoured out and carried along by the current, and this process continues until equilibrium is produced by the fall of the flood. Then with the consequent decrease in the velocity, the silt in suspension begins to deposit and fill up the bed, till eventually the normal low water bed level is again reached. This action takes place in all rivers, but is most marked in those which run in soil of a very light character. In bridging a river of this description the adoption of very deep isolated pier foundations, generally associated, in consequence of the expense of these foundations, with wide spans, is advisable, if practicable.*

(13) In the design under review the deep foundations are rightly provided but owing to the arched type of superstructure adopted, the spans *have to* be limited in width, resulting in the foundations of the piers being too close together for economy. If laid flat, the well foundations would form a continuous solid floor nearly 20 feet in depth. Consequently, in this and in all similar cases wider spans are a distinct economy, to effect which a steel girder superstructure is a necessity. Such an aqueduct would not cost more than a railway bridge of similar span and width, because the load carried although greater in amount than in the former case, is practically entirely dead load, and the aqueduct would probably be even a lighter structure than a railway bridge subject to a heavy vibrating live load. By adopting moderate spans of, say, 120 feet, the expense of pier foundations could be

* But in cases like that at Nadrai where the flow is syphoned under a considerable head of pressure the floor of the syphon barrels should be of masonry or concrete and the bed of the stream both up stream and down stream should be strongly protected with heavy stone riprap or concrete blocks supported by curtains of masonry wells or sheet piling.—ED.

reduced by one half or nearly so while on the other hand the safety of the work would be considerably increased by the lessened obstruction of the waterway and consequently diminished scour which would be decidedly a desideratum as a 30 feet deep scour previously alluded to can by no means be regarded with equanimity even with the 50 feet deep foundations adopted The length of the aqueduct piers with a girder superstructure would have to be increased as the canal water section would be split up into several channels each separated by the girders the vertical box plates of the lower flanges of the latter being carried up to above full supply level to form the sides of the channels the rest being open truss work With eight spans of 120 feet the available waterway above normal bed would be $8 \times 120 \times 22 = 21{,}120$ square feet The velocity of passage before the bed is scoured out would then be $\frac{140{,}000}{21{,}120} = 6·6$ feet per second and at 10 feet lower level $\frac{140{,}000}{30{,}720} = 4\frac{1}{2}$ feet per second nearly This allows of some increase in the width of the waterway as executed which is evidently desirable

(14) This increase in width must however be limited so as not greatly to exceed the normal width of the river channel Scouring action in the bed of a river cannot be prevented by greatly widening the natural channel and lengthening the bridge If this is done in the mistaken idea that the natural scour of the bed would cease owing to the provision of ample waterway laterally the certain action of the first falling flood would be to deposit silt on both sides thus reducing the channel to its normal proportions In future floods this lateral deposit would either remain undisturbed the channel reverting to its normal regimen—or else the current would bore it out on one side heaping deposit on the other thus causing great danger to the work from cross-currents This property of rivers is well known and in the Punjab for this reason the bridges over several large rivers have had to be curtailed in length and the channel restricted by protective works into a straight comparatively narrow reach more in accordance with the normal channel

(15) We will now proceed to review the details of this work The arch is $3\frac{1}{2}$ feet thick at the crown or $·42\sqrt{r}$ increasing to 4 feet at the springing

The thickness of the piers at springing is 7 feet or $·116S$ or $\frac{S}{8\frac{1}{2}}$ * As $\sqrt{S}$ the thickness would be 7·7 feet The piers widen out in a curve to 9 feet at the base The spandrels of the arches are lightened by a series of jack arches supported by the longitudinal piers shown in Figs 3 and 3a The parapets are corbelled out to form the roadways on either side One provides a 12 feet cart track and the other a 6 feet wide footway The corbelling and arching is ingeniously arranged but it takes up a great deal of

* By the rule ($t = 1 + 0·8\sqrt{r}$) suggested in par (4) it would be 7·2 feet —Ed

room, necessitating a lengthening of the arch, piers and abutment, which could be reduced, if no regard were paid to architectural features

The wings consist of a pair of curved landings ending in the canal earthen banks, which are here of semicircular shape to receive them On the river side are two large water wings, curving a full quadrant and continued to well beyond the termination of the land wings, forming a very efficient protection to the flanks of the work

The discharge of the canal is 4,100 cubic feet per second, which has a velocity of 4 feet per second in passing through the aqueduct, the curves narrowing the canal are not shown on the plan, the ordinary bed width of the canal is 230 feet The banks on each side of the narrowest part are revetted, and enclose a puddle core of large dimensions

(16) Fig 4 contains the plans of the Gunneram Aqueduct over a branch of the River Godaveri, taken from the "Madras Irrigation Manual" It has forty-nine spans of 40 feet, and the width between parapets is 23½ feet, with a depth of water of 6¼ feet The length of this structure, which is 2,250 feet between abutments, or nearly half a mile, is such that some slope is necessarily given to the canal bed, this was originally made 24 inches, the spring line of the arches being built lower in each span The surface level of canal now * shows only a slope of 7 5 inches (*vide* sections on *AA* and *BB*), in 2 200 feet

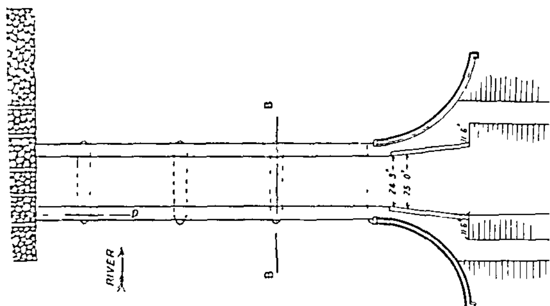
The river flood has risen as high as R L 23 91, *i e*, nearly 24 feet above bed and actually as high as the surface of the canal supply in the aqueduct (*vide* section on *AA*) † This must have subjected the work to a very severe test and clearly proved the sufficiency of the shallow foundations adopted The piers and abutments are built on circular wells 6 feet in diameter and 7 feet deep leaving an interval of 3 feet to floor level, which is occupied by the spreading bases of the piers A continuous floor of pitched rubble 3 feet deep protected by a series of horizontal curtain walls founded on shallow wells the whole 6 feet deep, is provided The existence of this floor is what undoubtedly prevented the pier foundations from being undermined during the exceptional flood alluded to Deep reinforced concrete sheet piling fore and aft of the floor would, however, make a better protection with mass concrete filled in between

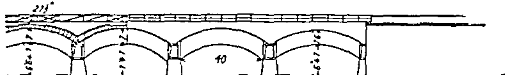
Owing to the great difference ‡ in the material of which the respective river beds are composed, no useful comparison between a work which is the exponent of shallow foundations and the Nadrai Aqueduct can well be made Under the conditions prevailing in the case of the Kali Nadi, shallow

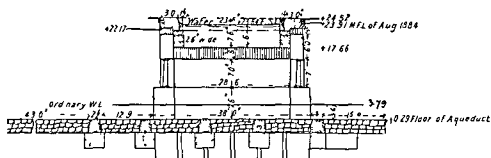
* The fall was reduced from 24 inches to 8 inches in the year 1883 by raising the down stream end and the increased pressure at that end caused the accident of August 1884 — ED

† … bout 26 feet … otection of

‡ … n the tidal influence of the sea and so has a deeper water cushion down stream The real difference however between the two structures in point of stability was the substantial floor and bed protection at Gunneram and the practical absence of such at Nadrai —ED







Godaveri River Forty nine Spans of 40 feet

[To face p 300

foundations as have proved so successful in the Gunnerum Aqueduct would in all probability be a failure owing to the great velocity engendered and the non-resisting* character of the material of the river bed unless they were continued for a very long distance below the work and reinforced by sheet piling. Such a construction, though it would interfere considerably with the regimen of the river, would doubtless stand, but its cost would probably be equal to that of the deep foundations adopted, in addition to which the point of dangerous scour would only be transferred lower down the river bed.

(17) As regards detail, the parapets and spandrel walls are remarkably slight, being only 2 feet thick. The spandrels are, however, filled up solid with concrete to the level of the intrados of the arches. The widening of the parapet to form a roadway is effected by external arches springing at a higher level, which level is horizontal throughout, not on a falling incline as is the case with the main arches. These arches are carried by stone columns built on the cut waters. This projection is 3 feet wide, allowing a total width of 5½ feet for the road way on either side. The thickness of the arches at the crown is 2 feet 9 inches or .45 $\sqrt{r}$.

The piers are 6 feet wide at the spring line or .15 S and widen out to 7 feet at bed level and to 10 feet at top of the well foundation. These appear to be in good proportion, $\sqrt{S}$ being 6.32 feet.† The section of the abutment is very light, due credit being given no doubt to the large buttresses in the rear and to the buttress effect of the wing walls.

These abutments with 40 feet spans are of four-fifths less sectional area than those of the Kern Aqueduct with only 20 feet spans, the great thickness of which has been already noticed in para. (8).

The disposition of the wings is peculiar. There are two large concave level crested wings which start from the outside of the abutments, forming water wings, inside which is the earth of the embankments, while the water is apparently contained by two inner walls, which are not shown to have any foundation, and may be just vertical walls of dry masonry.

This arrangement is probably due to there being no natural banks to the river, the canal embankment itself confining the river within the bridge abutments and these run at right angles to the axis of the stream. Without a cross section at this point the exact arrangement of the junction of the earthen bank with the masonry of the bridges cannot be ascertained, the plan over all being certainly defective. This work is contemporary or nearly so with the Solani Aqueduct on the Old Ganges Canal, having been finished in 1852.

(18) Figs. 5 contain the plans of the Kesarapali Aqueduct on the Ellore Canal, Madras. This work is rather remarkable in having a clear overfall of 12 feet in the bed of the drainage line; the design thus resolves itself into a bridged fall, with the bed of the canal below that of the upper reach of the drainage line.

* See note on previous page.

† By formula ($t = 1 + 0.8 \sqrt{33} = 5.7$ feet).—Ed.

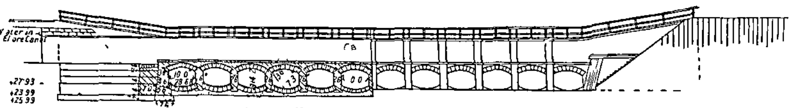


Fig 5 —Kesarapali Aqueduct Ellore Canal

Figs 6, 6a, 6b, 6c —Kao Nadi Syphon Aqueduct, Sôn Canal

A depressed bed involving a so-called syphon has been avoided, which is always to be effected if practicable, as syphons are very liable to fill up with detritus and cause a breach in the canal Another marked peculiarity in this design is that the crests of the parapet of the aqueduct are purposely kept only just above F S L , this is to enable surplus canal water to spill over either side of the aqueduct into the drain The work then fills the *rôle* of an escape as well as an aqueduct

The openings are 10 feet wide by $5\frac{1}{2}$ feet high, and are provided with inverts which take up a great deal of waterway and could well be dispensed with, the floor being made thicker, if necessary There is little or no upward pressure on the roof, so that arched vaulting is suitable

(19) Figs 6, 6a and 6b are a representation of the Kao Nadi Syphon Aqueduct carrying drainage underneath the Son Canal Both halves of the work are exactly similar In order to save headway the tunnels are vaulted with flat ashlar slabs, which are held down by bolts passing through the piers and are covered with a layer of brickwork and concrete A 10 feet roadway is provided on each side, consisting of slabs of stone resting on stone corbels

(20) The flood level of the outside water is R L 330 , that of the underside of the slab roof is 319 There is consequently a head of 11 feet of water acting upwards, tending to lift or break the roof when the canal is empty When the canal is full and the drain empty, the weight of water pressing on the slabs is that due to $9\frac{1}{2}$ feet depth of water The former pressure is the greater, but in calculating the thickness required for the slabs the weight of the slabs and the superimposed masonry and concrete is in their favour, whereas in the latter case it is against their resistance to rupture, so that the slabs are subjected to a uniform load of, say, 10 feet of water plus their own weight Taking this at a specific gravity of $2\frac{1}{4}$ and the thickness being $2\frac{1}{2}$ feet the load is equivalent to a weight of $10 + (2\frac{1}{2} \times 2\frac{1}{4}) = 16$ feet of water, weighing $\frac{16}{36} \times 20$ cwt $= 9$ cwt per square foot, the span, or L, being 6 feet, or 72 inches, the width B being 12 inches To find the required depth using a factor of safety of 4, the breaking weight, or W, is 36 cwt per square foot Then the following formula for strength of rectangular beams uniformly loaded, taken from ' Molesworth's Pocket Book," p 136, will state the case, viz , $W = \frac{8KBD^2}{L}$ This involves two unknown quantities, D and K, of which K, *i e* , the coefficient of rupture, must be obtained by experiment This is easily done by having some beams, say 1 or 2 inches square, cut off the stone to be used, laid out on supports at a convenient distance apart, say 4 feet, and loaded in the centre by a hook passed over carrying a weighing platform on which weights are placed by degrees till the beam breaks The weight of the platform, etc , should naturally be included in the count Then K, or the modulus of rupture, will equal $\frac{LW}{4BD^2}$ either in inch lbs or inch cwts as W is taken in lbs or cwts In this case L is the

l[illegible] between supports of the beam operated on, W the average breaking weight of several experiments, and b and D the breadth and depth of the beam [illegible]. Having thus obtained a reliable value for K it can be substituted in the formula $W = \frac{SKBD^2}{l}$ whence W being the [illegible] distributed load on the slab 1 foot wide in cwts or lbs, B being the breadth 1 foot or 12 inches and l the length between the piers likewise in inches, D the required depth will be $\sqrt{\frac{lW}{SB}}$.

We have seen that the upward pressure below the slabs is that due to a head of 11 feet of water; from this should be deducted the weight of the slab and superimposed material equal to a corresponding weight of water per square foot of 2[illegible] [illegible] 6 feet. The balance of head acting will then be 11 − 6 = 5 feet of water or [illegible] tons per square foot. This will have to be taken by the tie rods. Supposing the ties to be 5 feet apart the tension on each will be 5 × 6 ÷ [illegible] = 4 tons nearly (the length of the slabs being 6 feet) requiring a sectional area of [illegible] square inches or 1⅛ inches diameter.

(21) In this example the levels of the inlet and outlet of the syphon are the same, hence the water will be higher one side than the other to allow for the increased velocity which is generally allowed for in designing these works.

The discharge of the velocity through a syphon or culvert is determined* by the use of the Chezy formula $V = 100c\sqrt{RS}$ or $100c\sqrt{R\frac{h}{l}}$ using Kutter's coefficients. In the latter expression h is the head, l the length of the barrel. From this $h = \frac{V^2}{(100c)^2} \cdot \frac{l}{R}$.

The value of c is obtained from tables of Kutter's coefficients, that of h being assumed and subsequently corrected. The coefficient is but slightly affected by the value of S, the hydraulic slope, but is mainly a function of R the hydraulic mean depth or of $\frac{1}{\text{H.P.}}$, *i.e.* the area ÷ wetted perimeter.

To the value of h thus obtained (v being given) must be added the loss of head at entry which is $h = \left\{\left(\frac{1}{c_1}\right)^2 - 1\right\}\frac{v^2}{2g}$ (a)

In this the coefficient c_1 is that appertaining to the velocity formula of a jet given in par. (1) Chap. V, and may be said to vary from .62 upwards; v is the velocity in the syphon barrel.

A selection of the proper coefficient will have to be made from the list in par. (8) Chap. V, modified by the shape of the barrel according to the table given in par. (7). Thus for an oblong shaped vent a higher coefficient should be adopted than for a square one.

As a general rule .8 is an average value for c if the orifice is square

* More usually by the method indicated in Chap. V (67).—Ed.

$\left\{\left(\frac{1}{c_1}\right)^2 - 1\right\}$ in this case would be ·5625. If the width is twice the depth c will be increased according to par. (7) Chap. V by 1·03 or become ·824 and the expression $\left\{\left(\frac{1}{c^1}\right)^2 - 1\right\}$ will be ·456. A value of ·5 is often adopted.

With velocity of approach taken into consideration the above formula should read approximately $h_1 = \left\{\left(\frac{1}{c_1}\right)^2 - 1\right\}\frac{v^2}{2g} - \frac{v_1^2}{2g}$, v_1 being the velocity of approach.

The value of h to produce the velocity v in the syphon should also be reduced by that of the velocity of approach which, not strictly accurate, is sufficiently approximate for practical use.

(22) A practical example will now be worked out, one barrel of the Kao Nadi Syphon, Fig. 6, being taken. The area of this is 36 feet, the wetted perimeter 24, consequently $R = 36 \div 24 = 1·5$. The velocity adopted will be 8 feet per second, in this case $\frac{v_2}{2g}$ = unity. The discharge will then be $36 \times 8 = 288$ second feet. The length of barrel (l) is 104 feet. It is required to find the head to produce this velocity. As before we have $h = \frac{lv^2}{(100c)^2 + R}$.

Here the value of c for $R = 1·5$ and $S = 1$ in 1,000 with n taken at the usual value of ·013 is, according to Table XII, Part II, p. 142, Hyd. Manual, is 1·249, say 1·25. The true value of S or of $\frac{h}{l}$ in this case will be much in excess of 1 in 100, being more like 5 in 1,000, but in Jackson's tables slopes steeper than 1 in 1,000 are all accounted as of that slope, as the coefficient varies but slightly. h will then be $\frac{104 \times 64}{(1·25)^2 \times 1·5} = \frac{6{,}626}{234·375} =$ ·283 feet.

To this we must add loss of head due to entry, or

$$\text{with } c_1 = ·8$$
$$h_1 = ·5625 \times 1 = ·562$$

Total ·84 feet.

The head due to velocity * of approach, assumed at 3 feet per second, will be $3 \times ·01555 = ·04$ feet. If this be twice deducted the head required would be reduced from ·84 to ·76 feet. A somewhat easier method with same result is by use of formulas (23—26), par. (67), Chap. V.

(23) In designing a syphon in which a natural stream or river is carried underneath a canal, care must be taken that the channel below the work is of sufficient slope and sectional area to carry off the accumulation of water

* Head due to velocity of approach does not enter into calculations of discharge through syphons, as the dynamic head is changed into static head on change of direction of flow at the syphon.—ED.

that is backed up above the work, as for a time the channel will be called upon to carry a discharge in excess of the maximum flood discharge; this involves the straightening or widening of the tail channel.

(24) Referring again to Fig. 6, the disposition of the wings and the narrowing of the waterway of the canal is shown in the general half plan (Fig. 6b). The land wings, as usual, are curved on plan, which in the case of aqueducts is undoubtedly the only suitable form, while two straight flank walls with sloping crests enclose the cistern at the head and tail of the syphon. The vertical wall at the exit end is a bad form, as it hinders detritus being washed through the syphon, necessitating its removal by excavation. The exit end should not be a dead wall, but an easy incline, as shown by the dotted lines in Fig. 6. In most modern designs this sloping approach is provided at the inlet end as well as the outlet of the syphon, as exemplified in Fig. 9, *post*, though it is not absolutely necessary at the upper end.

(25) The drawback to a slabbed culvert of this description is the limit it imposes on the span of the openings and the great expense of the ashlar slabs.

A far better construction would be to form a flat concrete roof supported by rolled iron beams, or one of reinforced concrete; the span could then easily be increased to 10 feet, and the cost would be very much less.

Fig. 6c shows the construction.

The beams are 10 feet span and 5 feet apart. The weight on them will be equivalent to 15 feet of water, or the total distributed load $= 10 \times 5 \times 15 \times \frac{20}{36} = 417$ cwt, the weight of a cubic foot of water being $\frac{1}{36}$ of a ton.

From the Table given in "Molesworth's Pocket Book," p. 146, a beam 12¼ inches deep with 6-inch flanges and ⅜ inch web will safely carry this load.

Cross beams of small section will be laid along the centre of the span between each main girder, thus dividing the platform into 5 feet squares. The total depth of the roof can be reduced from 2½ feet to 1½ feet, giving an additional 1 foot in the headway of the sluices.

(27) A very recent instance of a design to pass a canal under a river is illustrated in Fig. 8 of the Burra Bubsa Syphon, Trebeni Canal.

This is a better design than any hitherto given, the main point in betterment being the double approach slope provided. This is obviously the best arrangement, as it encourages the scour of material, which in the case of the Kao Nadi syphon must be left in the barrels or in the approach cisterns, and require clearance possibly several times in the year. The greatest possible head acting on the roof of the barrel is about 6 feet. The counter-weight is only 2¾ feet depth of masonry at specific gravity 2, equivalent to 5½ feet of water. This leaves a little over ½ foot of water pressure unbalanced, producing tension in the platform. In this calculation the drop in the water surface or hydraulic gradient is allowed for.

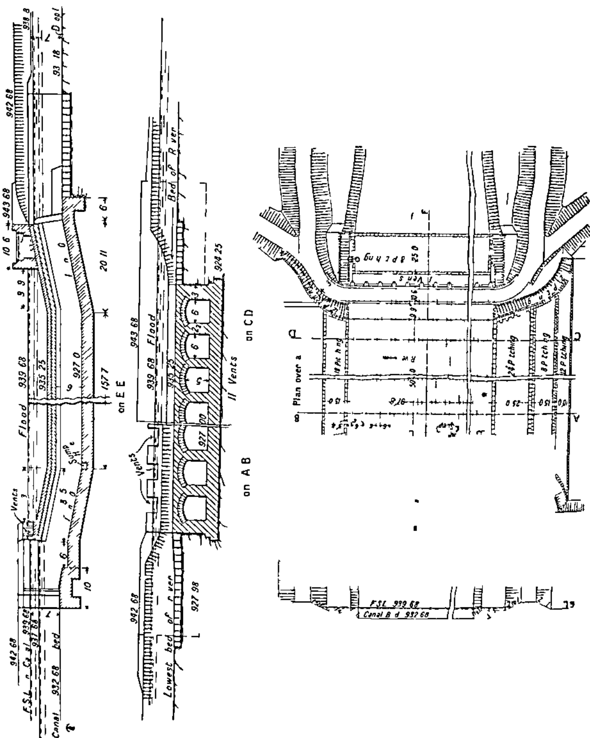


FIG 8 —Burra Bubsa Syphon Trebeni Canal (Bengal)

A peculiarity in this design is that the F S L of the canal and high flood in the river are at the same level On the canal side up stream the parapet of the superpassage is provided with vents 4 feet by 4 feet by which water can be allowed to fall into the canal syphon or else the canal can be allowed

parapets 400 feet. The arches had to be raised well above the canal bed on account of the exigencies of navigation and also of the level of the bed of the torrent. The 30 feet wide spans are therefore suitable. The thickness of the arches at crown is ·45 $\sqrt{r}$. The thickness of the piers at springing is 6 feet or ·2S. We have already seen (Chap. IV, par. (12)) that a proportion of ·175 S giving a thickness of 5¼ feet is sufficient.* The bottom width could be retained at 8 feet, the piers having either a straight or a curved batter as has been designed. The foundations, which are probably on good soil, are admirable (*vide* par. (11), Chap. IV.).

The abutment, by the look of the section, is of ample thickness, a good feature in a work of this magnitude and description. The actual incidence of the resultant line of pressures on the base has been graphically found; the method of working, which has already been fully explained in Chap. IV, consists in first finding the centre of gravity of the half arch and its load of water, the latter reduced in depth, as shown by the horizontal dotted line, to an equivalent mass of masonry. This process is shown in Fig. 10b and the reciprocal funicular polygon above Fig. 10a. The forces 1 and 2 are the areas of the two halves into which the half arch has been divided. Having found the centre of gravity of the half arch, a horizontal line is drawn through the centre of the arch crown to intersect the vertical through this centre of gravity, and from the point thus found the line R is drawn in Fig. 10a through the centre of the arch at its springing till it intersects a vertical line through the centre of gravity of the abutment and its water load. In the force polygon (Fig. 10b) the load line composed of the areas 1 and 2, is continued down to measure 300 square feet, the area of the abutment with its load of water, and the line R is then drawn from the termination of 1 and 2 parallel to its reciprocal in Fig. 10a cutting the horizontal P at a point. From this point another line R_1 joining the termination of the vertical load line 1, 2 and 3 just obtained gives the final resultant R_1. This projected on the profile of the abutment in Fig. 10a from its proper starting point, viz. the last intersection found, cuts the base

* By formula ($t = 1 + 0{\cdot}8\sqrt{r}$) we get a thickness of 5·4 feet.—Ed.

(10a)

(10d)

(10c)

FIGS 10 10a 10b 10c 10d —Budk Superpassage S I n l Canal

of the abutment at a point some 5 feet within its heel. As no credit has been given to the weight of the earth backing with water above it, the centre of pressure on the base is probably well within the middle third.

It might here be mentioned that the calculation of the effect of buttresses in an abutment or retaining wall is effected as follows:—The wall should be considered as having a base equal to its normal thickness plus the length of the buttresses, but formed of two materials of different specific gravities, the solid portion being of the proper specific gravity of the material

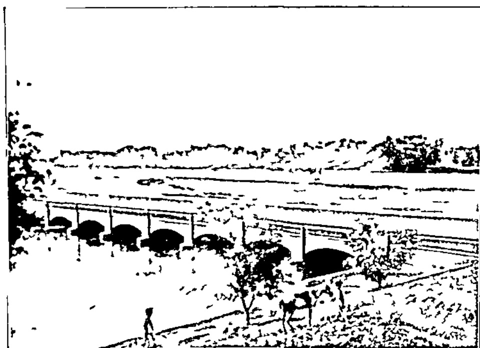
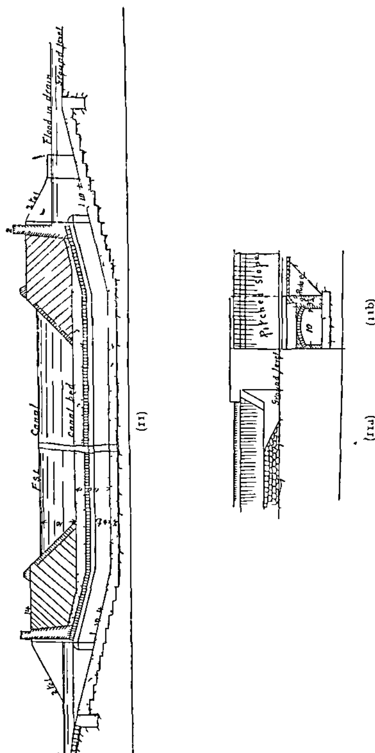


Fig. 10e.—The [illegible] Superpassage, Sirhind Canal.

and the part behind of a lighter specific gravity, equivalent to that of a material spread over the space of the same weight as the solid buttress only. Thus supposing a wall is 6 feet thick and is provided with buttresses projecting 4 feet and 4 feet thick and 6 feet apart, *i.e.* at 10 feet intervals, and let the specific gravity of the wall be 2, then the specific gravity of the 4 feet wide space behind will be $2 \times \frac{4}{10} = \cdot 8$, and the effective base width of the wall be 10 feet, not 6 feet.

The disposition of the wings is generally similar to that usually adopted in aqueducts, consisting of water wings as curved continuations of the faces of the abutments, and splayed land wings which carry parapets in continuation of those in the aqueduct proper. These wings are shown in the plan overall (Fig. 10c). As a further precaution the ends of the land wings are connected by a cross wall at bed level which apparently goes down to the full depth of the foundations. The land wings being in solid ground are stepped up in foundation, which is shown in the elevation Fig. 10d. A photograph of this work is given in Fig. 10e.



Figs 11, 11a 11b—Syphon on Chenab Canal

(31) Fig 11 shows longitudinal and cross section of the type design used for syphons crossing drainage, adopted in the Lower Chenab Canal It will be noticed that the canal banks are carried right across the syphon with

up to 1½ slope on the water side and a revetment wall outside. This is in ... for cart communication along the canal banks. This arrangement of a culvert two or three spans only in width is probably the most economical as it does away with the necessity of any land wings and additional arches to carry the roadway as adopted in Fig 7. The

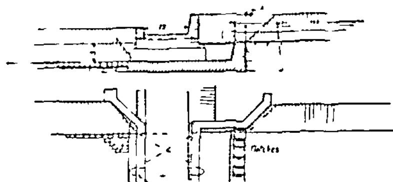
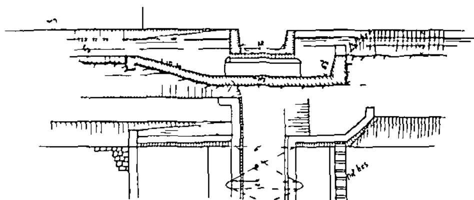


Fig. 12

upward head of water below the crown is 9 feet. To balance this a thickness of roof of $\frac{9}{\rho}$ is required, taking the specific gravity or ρ as 1.8 the thickness should be $\frac{9}{1.8} = 5$ feet. The actual average thickness of the arch and concrete above is about 4½ feet. There is therefore tension in the masonry


Fig. 13

equivalent to the weight of a layer of brickwork etc. of ½ foot in thickness which in lbs is $\frac{1}{2} \times 1.8 \times 62.4 = 56$ lbs per square foot. No doubt the cohesion of the mortar in the concrete above is more than sufficient to withstand this pressure, but in all masonry works, particularly those in connection with water, any tension should be avoided.

The general arrangements in this design are simple and excellent. The floor is only 2 feet thick, and the whole is made entirely of concrete. The proportionally wide spans are also a good point, as the wider they are the

higher the coefficient of discharge will become The double slope is a good arrangement for drainage lines as it gives an impetus to the water and tends

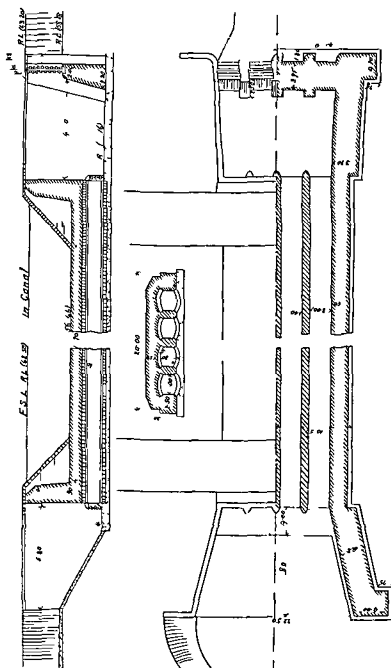


Fig 14 —Sohagiah Canal Syphon (Egypt)

to carry detritus through the culvert A direct overfall on the up stream side on the other hand absorbs a great deal of the velocity of approach The form of vertical drop above and slope below is suitable where a canal is taken under a drainage line as being more economical

piers 3¼ feet thick which is somewhat excessive The thickness of the roof is 2 metres or 6½ feet which is sufficient for a head of 13 feet The flood level of the drain is not shown but from the height of the banks it is probably about R L 61 at least That of the intrados of the culvert arch is 54 50 the head then will be 6½ metres or 21½ feet The syphon is evidently not designed to run full with the canal empty as it undoubtedly should be This state of things is mentioned by Sir Wm Willcocks in "Egyptian Irrigation" many of the older works in Lower Egypt having been designed on the hypothesis that the canals would never run dry, i e, the supply would never be cut off from the head Now however that head regulation is possible these works are not equal to the pressure brought upon them and this doubtless is one of them *

The slope of the earthen banks is carried through the work The distance apart of the parapets of the aqueduct is no less than 350 feet and it carries 20 feet of water

(35) Fig 16 is a type section of a syphon under a distributary from the "Madras Irrigation Manual" As will be seen both earthen banks are carried entire across the syphon

(37) In Egypt of late years wrought iron pipes have been used for both aqueducts and syphons The following is a description of them from Egyptian Irrigation —

* Syphons of the kind shown in Fig 13 are objectionable as tending to silt [illegible] —E [illegible] fall of water surface through them is slight and consequently also the velocity [illegible] would be appropriate in [illegible]

† Reinforced concrete tubes 10 or 12 feet in diameter this —E D

"They are generally constructed of ¼ inch sheet iron, butt jointed, *stiffened with angle irons at every alternate joint if over 12 feet in circumference*, and lap jointed if under 12 feet Since the sheets in the market are 8 feet by 4 feet, or 6 feet by 3 feet, the pipes are always constructed with their circumference some multiple of the length or width, so that there may be no cutting of plates The pipes are sometimes laid on a bed of concrete, varying from 1 metre to 25 centimetres in thickness, according to the quality of the soil, or they are laid on the hard clay soil and well pitched

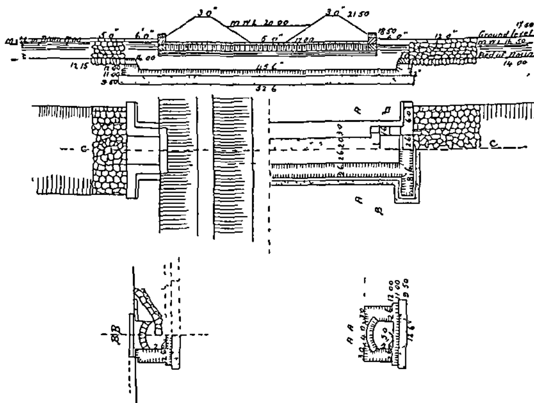


Fig 16 —Canal Syphon (Madras)

round with clay balls Where the pipes are used as aqueducts they are generally supported on wooden trestles The great advantage of using wrought-iron pipes is that they can easily be transported, they do not need expensive supervision during construction, and can be put together so rapidly that the cost and trouble of a diversion for the canal during the time of construction is avoided These pipes can be closed at the ends and floated to their destination By dredging the foundations where they have to be laid they can be floated over the site and then sunk without shutting the canal head"

A pipe of 16 feet circumference will discharge 300 cubic feet per second, with a head of 2 metres or 6½ feet, and 156 cubic feet with a head of half a metre or 1 6 feet One of 12 feet circumference, 180 cubic feet per second, with a head of 2 metres, and 87 cubic feet with a head of half a metre

(38) Where the drainage is slight inlets can be provided they simply consist of a small fall or pitched rapid protected by flank walls which conducts the drainage water into the canal the floor and banks of the latter being pitched all round to prevent damage If the inlet occurs at the roadway bank it has to be bridged A level crossing consists of an open inlet on the side of the torrent a regulating bridge across the canal and another at right angles across the exit of the torrent which is generally provided with a fall to expedite the speedy removal of the unwelcome guest It is very essential that quick-acting falling gates be supplied to the outlet bridge, draw gates would be much too slow in manipulation The balanced pivoted gates recommended for the Kosheshi Escape would, it is believed, answer admirably for such a purpose

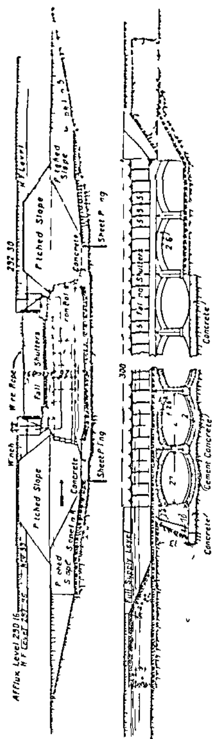


Fig 18 —Thapangaing Aqueduct and Level Crossing (Burma)

(39) Cases in which the passage of drainage across a canal is beset with the following difficulties are enumerated below —

(1) The drainage is too extensive to be wholly admitted into the canal

(2) It cannot be taken underneath in a syphon drain owing to the presence of the backwater of the parent river when in flood which comes up to the very canal bank hindering the free discharge

(3) It thus must be disposed of by a superpassage, but this involves the formation of a large deep reservoir on one side of the canal, which has to fill up before sufficient elevation is attained to admit of surplus passing at a high level over the canal

This problem was satisfactorily solved in the case of the Ali Superpassage and concomitant works in the third or fourth mile of the Agra Canal The works constructed consisted of firstly a solid embankment on the upper side of the canal which crossed a large depression, secondly a

water tower built some way off the canal in the depression provided with external sluice openings at different levels thirdly a culvert which connected the base of the tower with an inlet into the canal itself lastly an iron girder superpassage with a masonry fall in the further (river side) bank This arrangement has answered well When the depression fills up to a moderate extent all its water can be gradually drawn off through the tower into the canal When however as happens not annually but occasionally the tributary streams bring down so much water as to quite fill the reservoir the surplus is then disposed of automatically by passing over the superpassage into the river When the crisis of the flood is over the deep reservoir

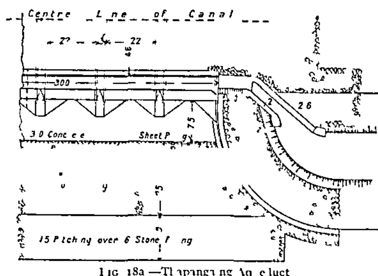


FIG 18a —Thapanging Aqueduct

can be gradually tapped into the canal by means of the water tower and inlet culvert Unfortunately no plans are available

(40) The subject of this chapter cannot be considered as exhausted without the insertion of the plans of the Thapangaing so called aqueduct Figs 18 and 18a contain the plan of the remarkable work which is really a combined syphon and level crossing The work was originally designed as a syphon to carry a hill torrent underneath the Mandalay Canal During its construction an immense flood came down of an estimated discharge of 57 000 second feet The previously estimated flood discharge on which the design of the syphon was based amounted to 24 000 second feet consequently the design had to be remodelled In effecting this it was wisely decided to leave the syphon as originally projected but to supplement it by a level crossing The parapets of the canal superpassage were removed for 300 feet and a series of wooden falling shutters substituted There are sixty pairs of these each being 5 feet wide by 7 feet high The width of the canal here is 46 feet Each pair is connected across the canal by a wire rope which passes over brackets raised above the shutters and side tension rods prevent the shutters from falling inwards The tension rods on the upstream side are held in position by a let go gear and when released both

shutters fall one towards and one outwards allowing the flood to pass over them. The canal head must previously be closed and the water emptied all but [illegible] feet to act as a cushion.

This arrangement has been twice tested and found to act well. A successful precedent of novel construction is always most useful as it enables similar works to be designed with confidence and with possible improvements on the original.

The general alignment of the wings (Fig. 18a) appears to be excellent. There are the quadrant outer wings affording a good approach to the syphon, while the splayed inner wings connect the canal banks with the walls of the upper passage. A central wall is introduced here with the object of allowing the level berm to be continued between it and the canal wing up to the abutment. The head of water acting below the soffit of the arch is 13 feet, to meet which is 4.6 depth of arch and pitching equivalent to 9 feet of water; consequently holding down bolts have been inserted to assist in meeting this unbalanced hydrostatic pressure of 4 feet at the crown. The sloping approaches to the syphon are good and worthy of imitation.

(41) A view is given of the Dhanauri Level Crossing on the old Ganges Canal in Fig. 19, for which we are indebted to Mr. Buckley. There is

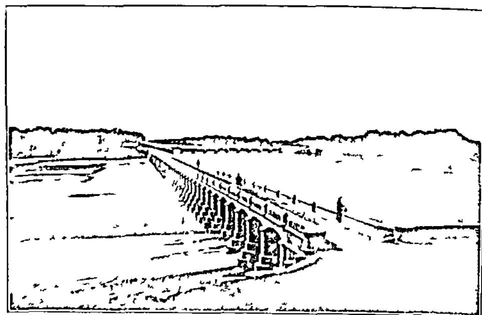


FIG. 19.—Dhanauri Level Crossing, Ganges Canal.
(In the foreground a barrage across the torrent; in the background the barrage across the canal.)

another level crossing of the Somb River or Torrent across the Western Jumna Canal at Dadupur. This is still working well though built originally nearly a century ago, of very simple design.

(42) Several important level crossings were built on the Upper Jhelum Canal (of the Punjab Triple Canal Project) during the period 1905—1915 They all consist of barrages built across the torrents, with spans of 40 feet, controlled by gates of the 'Stoney" live-roller pattern, the canal waterway being similarly controlled with gates of 25 foot span The gates have to be very rapid in their working as the torrents are liable to rise in floods of high intensity within half an hour of the commencement of heavy rainfall

(43) But the most important level crossing hitherto built has been that across the River Ravi, at Balloki forty-five miles down stream of Lahore The Upper Chenab Canal, of the Triple Canal Project, tails into the river on one side just above the barrage whilst the Lower Bari Doab Canal of the same Project, off-takes from the other side of the river above, the barrage This level crossing has been built instead of the Ravi Syphon, which had been originally designed but over whose design and estimate of cost much difference of opinion arose The Ravi Level Crossing, at Balloki, was completed in the year 1913

EDITOR'S NOTES.

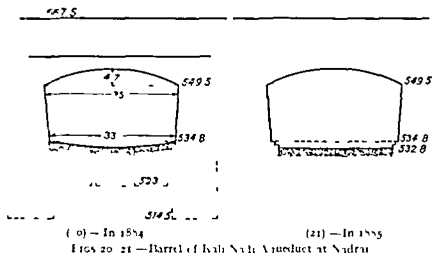
(Q) *The Kali Nadi Aqueduct at Nadrai*—The Lower Ganges Canal, at Mile 34, passes over and across the Kali Nadi stream by an aqueduct at Nadrai When this work was being first designed by Captain Jeffreys in 1871, he supposed that the maximum flood of the stream at that point would be 26 400 cusecs, but he proposed to allow it a waterway sufficient for 36 000 cusecs, at the rate of 12 per square mile over its catchment area reckoned at 3 025 square miles To pass this with a mean velocity of 10 feet per second he proposed to allow seven spans of 35 feet, 15 feet high to springing of arches Later, he noticed that half a mile below the proposed site of the aqueduct, there was an old bridge across the Kali Nadi, which had been in existence for at least 100 years This bridge had a total waterway of only 1,146 square feet, and the flood marks on it showed that it had passed floods with a maximum ponding up of 1 5 feet From the formula $D = ca\sqrt{2gh}$ $= 75 \times 1146 \times 8\sqrt{1\ 5}$, he reckoned that the maximum flood would be 8,450 cusecs, with a mean velocity of 7 33 feet per second There was another bridge thirty miles up stream of Nadrai, from flood marks on which Jeffreys reckoned that the maximum flood had been 4 220 cusecs from a catchment area of 1,556 square miles above that point These data led him to conclude that the maximum flood of Nadrai would not exceed 9 500 cusecs, at the rate of 3 66 per square mile of a catchment area of 2,593 square miles Doubling this, for safety, he proposed to allow for a maximum flood of 18,000 cusecs The aqueduct was designed with a lower waterway of 5 spans of 35 feet, as shown in Fig 20, or 633 × 5 = 3,165 square feet

Captain Jeffreys recognised that this allowance of waterway was open to criticism, but he explained "I cannot, with this 'Mogul' bridge of more than a century old staring me in the face, recommend a greater provision than that [illegible] rway of 2,450 square feet, that [illegible]

And ([illegible] the design, with the

remark Although the flood volumes given by Major Jeffreys are perplexingly small yet there seems no reason to doubt their accuracy

The work was built accordingly the masonry channel for the canal



(20) — In 1854 (21) —In 1885

Figs 20 21 —Barrel of Kali Nadi Aqueduct at Nadrai

being made 102 feet wide to discharge 5 374 cusecs with a water depth of 9 4 and a mean velocity of 3 feet per second

The abutments and piers of the aqueduct were founded on masonry wells sunk to a depth of 19 feet (533 5 514 5) below stream bed but the

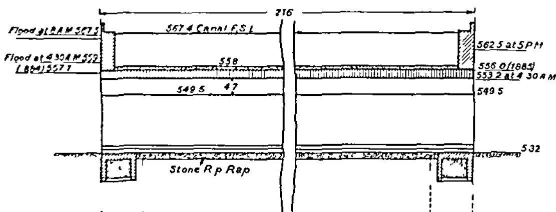


Fig 22 Old Kali Nadi Aqueduct destroyed in 1885

curtains were sunk 10 feet deep only the foundation soil being sand There seems to have been very little protection of the stream bed Under the spans there was a dished floor of block kankar riprap grouted with lime but little else and block kankar is a weak material incapable of resisting the erosion of high velocity currents or the pressures of an inverted arch In October 1884 a flood occurred which rose to 557 up stream and 553 5 down stream of the aqueduct and the local executive engineer (Mr W Good) estimated its maximum intensity at 37 000 cusecs with mean velocity 11 7

This flood washed out the block-kankar floor and undermined the foundations of the fourth pier from the right, which collapsed, bringing down the arches which rested on it Had there been suitable protection to floor and stream bed this accident would probably not have occurred

Nine months later another great flood occurred, and this time the aqueduct, which had meanwhile been repaired, was completely destroyed. *The flood began rising on the afternoon of July 16th, 1885 According to* Mr Good, who eye-witnessed the flood, the water levels were 559 up stream and 553 2 down stream of the aqueduct at 4 30 a m on the 17th, down to which time no damage had occurred At this time the mean velocity through the barrels was about 16 feet per second and the discharge about 50 000 cusecs " Immediately after this the up stream level rose 4 feet in one wave, and part of the old revetment over Arch No 1 fell in" This *sudden* rise of up stream water cannot have been due to increase of flood, and we may infer therefore that it was due to obstruction of waterway by the collapsed masonry

' At 8 a m the up stream level was 567 5 (within 0 5 foot of the top of the new right revetment wall) and surged over it in waves 10 to 15 feet high, and then the whole of this revetment and the rest of the arches collapsed At 9 30 a m the only vestige of the aqueduct was the mass of the left revetment The water ran smoothly between the revetments The water down stream of the aqueduct was in waves 20 feet high and at intervals of 100 feet, and laterally in a formidable swirl towards the left bank" Assuming that the condition of flow was that over a submerged weir with cill at level 532, and 200 feet long, and allowing a deduction of $\frac{1}{11}$ for obstruction by the fallen masonry, Mr Good calculated that the discharge at 9 30 a m was 94,000 cusecs "At 3 p m the left revetment, which had so long blocked the waterway, fell in, and the land wing followed By this time the flood had cut a channel 100 feet wide at the back of the left abutment, through the canal embankment, and another 50 feet wide on the right The flood reached its maximum level at 3 45 p m, when the up stream water level was 569 on the right bank" And Mr Good reckoned the discharge of this time to be 121,000 cusecs Thereafter the up stream levels sank, but the down stream level continued to rise till 5 p m, when it was 562 5 This suggests that the high up stream levels prior to 4 p m may have been due to invisible obstructions rather than to magnitude of discharge

The Superintending Engineer, Major Home, reckoning the stream as flowing over a broad-crested weir with cill at 532, and with waterway 275 feet wide (including lateral breaches) reckoned the flood to have amounted to 132 000 cusecs, and suggested re-building the aqueduct with eleven spans of 60 feet, to discharge with mean velocity = 8

The Chief Engineer, Colonel Forbes, suggested thirteen spans of 60 feet, but Colonel Brownlow, who was now Inspector General of Irrigation, thought it would not be safe to build with less than fifteen spans of 60, on the ground that the flood might well have been as high as 140,000 cusecs

After the accident to the old aqueduct in 1884, Colonel Brownlow had been hard put to it to explain why he had designed it to discharge only 18,000 cusecs, and we can well understand that the total destruction of the work by a still greater flood in 1885, whilst he was still apologising, decided him to think of "safety first," and nothing else that mattered

At any rate the new aqueduct was built with fifteen spans of 60 feet having a total waterway of 10,000 square feet between stream bed and arch soffits to discharge 140,000 cusecs. The design assumes that the high flood level will be at the same level as the canal bed (R L 558) that the waterway will increase through bed scour and that the afflux will be only 6 inches. This would seem to be in error seeing that the great flood of 1885 rose to 562 5 down stream of the old aqueduct. The *Vali* will be able to discharge only about 50,000 cusecs up to level of springing of arches (R L 546) as an open channel. As it rises above that level the mode of

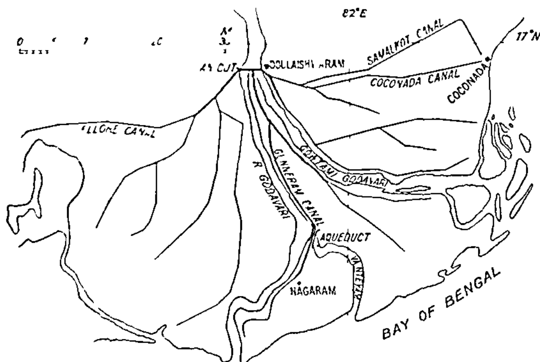


Fig 23 Delta of River Godavari

discharge will change from that of open flow to that of pipe flow and although the earthen bed will be free to erosion the rate of enlargement of waterway by such erosion is an uncertain problem whilst as soon as syphoning occurs a rapid increase of afflux to create velocity head and entry head $\left(1\ 5\ \frac{V^2}{2g}\right)$ will be on the cards and this added to a possible H F L of 562 5 down stream of the work may cause trouble up stream. The piers and abutments of the new aqueduct have been founded on masonry wells sunk to a depth of 50 feet below stream bed. The wisdom of this may be questioned seeing that a very substantial stratum of clay exists at a depth of 30 feet only below stream bed. As matters stand the foundations are composed of 268 wells with an aggregate vertical length of 15,019 feet or nearly 3 miles! By founding them on the clay at least one mile in length of wells might have been saved but the depth of 50 feet was arrived at as

the least at which, it was supposed that, under the most unfavourable circumstances, the pressure at base would not exceed the limit of 4 tons per square foot Colonel Brownlow advised fixing the limit so low as 2½ tons per square foot If the old aqueduct that was destroyed in 1885 had been designed as Colonel Sir Arthur Cotton designed the Gunneram Aqueduct on the River Godavari in 1851, *i e*, *as an aqueduct sluice*, with floor and apron and strong bed protection up and down stream, it might have remained intact, in spite of the great flood of 1885, even though the earthen embankments of the canal were breached through to right and left of it Not that one would deliberately design an aqueduct sluice, for choice, to discharge great floods by breach and overflow, but the new Nadrai Aqueduct would probably have been as efficient, if not more so, if designed with only ten spans instead of fifteen

(R) *The Gunneram Aqueduct on the Godavari* —The Gunneram Aqueduct carries the canal of that name (or Naggaram) across the Vainteyam branch of the Godavari Delta (Fig 23) It was designed in the year 1851 and built early in 1852 Colonel (afterwards General Sir) Arthur Cotton, describing the design in 1851, wrote ' This work is not a simple aqueduct, but rather an aqueduct sluice That is, the water, in floods, will rise above the crown of the arches The work is planned, accordingly, as a sluice, *i e*, with floor and apron The obstruction offered by it is about the same as that of the Anicut at Dowlaishwaram in the highest floods, *i e*, the proportion of waterway to the whole section of the river at that point is about the same as the corresponding proportion at the Anicut I have allowed for a front apron 10 yards broad, and a rear apron of 20 yards, supported by lines of wells along its whole length, with some loose stone also below them There will be a floor of large packed stone also under the arches, with a line of wells along the down stream side from pier to pier "

Colonel Baird Smith inspected the aqueduct in 1853, and wrote thus There are forty-nine arches of 40 feet Abutments and piers rest on wells 5½ feet diameter, 8 feet deep, in sand Over the well tops is a floor, which, in twenty five arches is of concrete The floor is one foot thick and supported by five rows of wells 4 feet in diameter, sunk 3 feet in sand The whole structure is of brickwork in excellent cement, the bricks being of the extraordinary size 18 inches by 6 inches by 3 inches, with a small hole ½ inch in diameter, through the middle, to facilitate baking The whole work was completed in three months from date of commencement of construction by Lieut Haig, R E

Within a few months (or possibly weeks) after the aqueduct was completed, a flood rose not less than 5 or 6 feet over the level of the tops of the parapets, burying the whole structure in water The height of this flood must have been about 30 feet The sectional area of such a flood, as provided by Nature, is approximately 72,000 feet, that provided by the engineers is 30 000 '

Supposing the mean velocity of the river flood had been from 6 to 8 feet per second at some distance from the aqueduct, this would have increased to from 15 to 20 feet per second past the obstruction, with afflux from 3 to 5 feet, but *owing to the work having been designed as a syphon, to resist such velocities, no harm resulted*

Colonel Cotton wrote in 1852 " That a single officer, with two or three overseers, should have managed 5,000 workmen with the help of only three or

f[illegible] r [illegible] cient workmen is one of the most [illegible] [illegible] met

Lieutenant Haig had spent Rs 1,60,000 on the work [illegible] estimated amount of Rs 73,000 only

In 1884 the longitudinal slope of the canal trough was reduced from 3 feet to 8 inches only and in this connection the side walls at the down-stream end were raised 3·75 feet higher They proved too weak [illegible] August 1884 a 100 foot length of the wall was pushed over by the [illegible] pressure The Madras engineers replaced this with a wooden [illegible] within ten days and carried on smiling

In October 1891 an extraordinary flood submerged the whole [illegible] Colonel Howley described the incident thus The flood of October went right over this fine old work about 18 inches deep in the middle to [illegible] at the ends The greatest difference of river water level above and below the work was 2·4 feet The only damage to the superstructure was washing away of some of the surface gravel and concrete of the right tow path and slight injury to the wooden railings The rough stone flooring in the river bed below the work on the left side was somewhat disturbed Sheet piling protection is necessary and the provision of a reserve of loose stone The flooring put down in the last three years along the front of the aqueduct has added much to its safety

This flooring was of concrete 1·5 feet thick 15 feet wide retained by a rubble masonry wall 2·5 feet × 3 feet founded on wells 5 feet square and deep

CHAPTER XII

RESERVOIRS AND TANKS

(1) When water is impounded for purposes of irrigation in a reservoir, this can either be effected by embanking a depression, which receives the run off of the rainfall from a catchment area, or else a defined river or stream is held up by a masonry dam or weir, or by an earthen bank In the former case there are innumerable examples in the small tanks that have been constructed in eastern countries where the slope of the ground is favourable Of the latter there are also many examples on a very large scale for instance the Bhatgarh Reservoir in Bombay, the Periyar project in the Madras Presidency, and the Assuan Dam in Upper Egypt These three works have already been referred to in Chap II , which treats of the sections of dams The large storage reservoirs in the United States are too numerous to be catalogued The Salt River Reservoir will impound $1\frac{1}{4}$ million acre-feet a record quantity

(2) In designing a reservoir, the first point requiring definite statistics is the maximum and minimum discharge to be expected from the catchment area the former for the purpose of designing the works for the disposal of the surplus water which passes through the reservoir after filling the latter to its full capacity , that is, the required length of waste weir or bye-wash or the discharging capacity of waste sluices, if such be adopted The second determines the capacity of the reservoir, which naturally must be such that it will fill in ordinary years, and on this depends the area of land which it is capable of irrigating and the revenue which may be expected to accrue

The subject of maximum discharge from catchment areas has already been investigated in Chap V , the minimum may be assumed as half the maximum, *i e* , with a maximum run off of $\frac{1}{4}$ of the rainfall, the minimum may be taken as $\frac{1}{8}$ or $\frac{1}{10}$ of the rainfall There are of course many instances of small rain fed tanks which naturally will not fill in a year of drought, though they may do excellent irrigation in ordinary years, so that in most cases the average rainfall has to be considered not the absolute minimum A tank however, situated on a stream which never fails to run intermittently during the rainy season, even in a year of drought, is naturally more valuable than one dependent entirely on local rain, which may be very scanty in some years

The ideal site for a reservoir dam is one across a narrow gorge in a stream or drainage line which above this point widens out in a long level and broad depression The discharge of the catchment may be such that the tank will fill more than once during the season According to the " Madras Manual of Irrigation,' the irrigating capacity of a tank thus supplied can be reckoned as $1\frac{1}{2}$ its actual capacity

(3) The actual capacity of a tank is the cubic contents of the water impounded between full tank level (F. T. L.) or F. S. L. (Full Supply Level) and that of the sill of the lowest irrigating sluice. One of the first points requiring attention after selecting the site for a proposed reservoir is the determination of its storage capacity up to different proposed levels of escape, i.e. to F. T. L. For this purpose marks should be fixed at differences of level of about 5 feet or 10 feet apart on any convenient short lines of section; the contours of these levels should then be marked out and surveyed all round the basin in order to obtain the perimeters and areas at each contour. From these the contents of each lamina can be calculated and the contents up to any contour. The same result can be obtained by a series of longitudinal and transverse sections taken up to the heights of the various contour levels. The former should be directed in conformity with the axis or axes of the figure of the basin and transverse sections at right angles to them. Should a winding river channel or depression form part of the basin it is often more convenient and correct to estimate its contents independently and add it in afterwards.

(4) The formulas in use for obtaining the contents or capacity from the horizontal contour areas are as follows :—

With two contour areas only, viz. A_1, A_2, at a common vertical distance apart (d). The contents $= \frac{1}{3}d\,(A_1 + A_2 + \sqrt{A_1 A_2})$.

If there be three equidistant horizontal sections the contents $= \frac{1}{3}d\,(A_1 + 4A_2 + A_3)$.

If there be any even number (n) of equidistant horizontal sections A_1, A_2, etc., up to A_n at a common distance d, the contents $= d\,(\frac{1}{2}A_1 + A_2 + \text{etc.} + A_{n-1} + \frac{1}{2}A_n)$.

The capacity of the reservoir being thus estimated, the amount of supply that can be expected annually from the catchment area can be obtained from the Tables given in Parts I and II of Table II and Part I, Table III, Jackson's "Hydraulic Manual," where also some excellent practical examples of their application are given, or else the run off can be obtained from Tables given in Chap. V of this work.

The irrigating capacity of tanks and duty of water impounded varies naturally with the amount of absorption and evaporation. In the Madras Presidency, where the system of irrigation from tanks and reservoirs is very extensive, the following is the quantity per irrigated acre required * to be stored to bring the crop to perfection :—

	Cubic feet	Acre feet
(1) Five months' monsoon rice crop	216 000	5 nearly
(2) Cold weather rice crop	175 000	4 ,,
(3) No. 1 if stored for next year's use	540 000	12.4 ,,

* Th

The reckoning of artificial supply includes all loss from evaporation and absorption in the reservoir and the irrigating channels This estimate, though rough, is useful as a guide For cold weather crops other than rice, such as wheat, etc , the storage in acre-feet will be about 2 The loss from evaporation varies with the climate of the country in which the reservoir is situated In Upper Egypt the maximum is taken at 39 inch per day, and that in India in the hottest and driest months may be anything between 0 5 inch and 1 inch , whilst it may be only 0 3 inch in the coldest month The annual loss by absorption varies considerably according to the nature of the bed of the tank, and may be taken, according to " The Irrigation Works of India," at one-half the yearly loss by evaporation

From experiments made in tanks in Rajputana, which has a dry, hot climate, the average daily loss from both causes was—October to March, 20 , April to June, 47 , and July to September, 41 inch per diem

This is equivalent to a total annual loss in depth of 6 15 feet from evaporation and 3 62 from absorption, giving a total of 9 77 feet

In Madras a continuous run of 1 cubic foot of water will irrigate 66⅔ acres of rice and double that amount of dry crops In new tank projects the duty per cubic foot per second is sometimes taken as—rice 50, dry crops 100 acres, inclusive of losses from evaporation and absorption *

(5) Whether a masonry dam or earthen bank is adopted to impound the water depends upon the local circumstances and comparative cost The usual limit in height of earthen banks used to be considered as 60 feet, but this has been greatly exceeded in some reservoirs in Bombay, where the Waghad Reservoir embankment is 95 feet high, and another is projected of a maximum height of 110 feet In England an embankment 125 feet high has been successfully constructed Now that the new system termed Hydraulic Fill has come into general use there appears to be absolutely no limit to height adopted The projected Oigawa Dam in Japan is to be 330 feet in height, exceeding even that of the Shoshone arched masonry dam which now holds the record at 310 feet

The works connected with a tank or reservoir consist of (1) the embankment or dam, (2) the waste weir, bye-wash or escape sluices for the disposal of surplus water, and (3) the outlet sluices for irrigation

(1) *Embankments*

(6) The common practice in Europe is to construct a puddle core in the centre of an embankment, the remainder of the material being composed of earth gravel and stones , in this case the imperviousness of the dam to leakage is entirely dependent on the puddle core

Where suitable earth is not obtainable to form the mass of the embankment, the adoption of a puddle core is obligatory, but where good soil is available the puddle core for embankments of, say, 50 feet in height is

* But it must be borne in mind that this water supply is supplemented by rainfall as noted —Ed

not necessary In India the use of a puddle core is generally limited to very large reservoirs, but even in the case of high banks it is often entirely dispensed with A thoroughly consolidated homogeneous embankment of earth is undoubtedly preferable to one of infirm material rendered water-tight by a puddle wall The large Waghad tank embankment, a section of

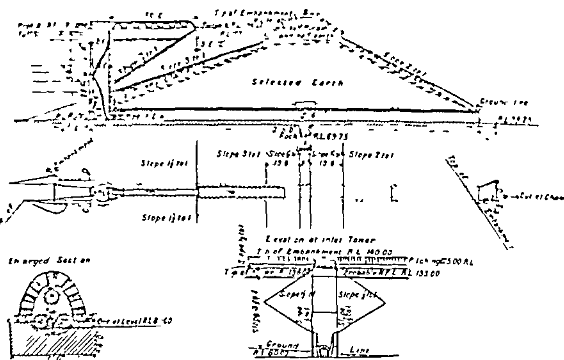


Fig 1—Waghad Tank Embankment

which is shown in Fig 1, was constructed with rammed layers of moist earth without a puddle core

(7) Unless clay soil is not easily obtainable near the site of the embankment, this adoption of a puddle core for small or large depths of water is not absolutely essential In forming an embankment without a puddle core it is obligatory that the earth be at least damp and consolidated in layers by heavy rollers The whole mass can be made of wet earth *i e*, practically puddled, if water is available The method of procedure is as follows — The bottom 10 feet of the base of the bank should first be thrown up and a temporary cut made in the solid ground or rock at a lower level to pass off waste water, or else the masonry outlet culverts can be first built and adapted for this purpose The work should then lie in abeyance until water has collected in the basin behind the bank The surface of the banks should then be divided into shallow basins about 12 inches deep by narrow partition walls of earth Into these enclosures the water should be pumped or baled up from the reservoir As soon as a series of these shallow basins are full of water, the earth is thrown in to fill them up level with the top of

the partition walls, after which another series of chequers are formed on top and again watered While one part is being filled up, another is being watered or chequered so that there is no intermission in the earth carrying When the embankment is thus raised the level of the bed of the escape cut can likewise be raised either by partially filling it up or cutting a new channel at a higher level so as to allow the water to rise to a further height behind the bank By the means thus described, each layer of earth is thoroughly soaked and clods dissolved so that no ramming or clod breaking is requisite, and the new layer is further consolidated by having 6 or 9 inches of water laid over it, the result being that the whole bank is composed of wet earth devoid of air spaces, which are inseparable if dry earth is used, no matter how much it may be consolidated by rolling or ramming Consequently when the tank fills, there can be no settlement whatever of the embankment If during this process the bank shows signs of supersaturation by bulging, the work should be suspended at this part for a day or two until matters adjust themselves and the layer of water subsequently reduced in depth

The author has constructed several embankments on this system, and not the least sign of settlement ever appeared in the bank thus formed when fully tested This system is probably best applicable where coolie labour is employed and earth carried on the head in baskets when it can be deposited wherever required

In many cases where stiff clay is the material, it is customary to mix a certain proportion of fine gravel with the clay, as gravelly soil is naturally far more watertight than pure clay, unless the latter is kept continually moist This can be done by arranging a certain proportion of the carriers to convey gravel, so that the mixing takes place automatically In the United States the universal practice was to construct the core walls of masonry but they are now quite discredited The embankments there are thrown up in layers of damped earth and rolled by heavy rollers Now, however the hydraulic method is superseding the ordinary system

(8) Undoubtedly by far the best method of forming reservoir embankments is that of ' hydraulic fill ' By this means immense embankments not only for reservoir but for high railway banks, have been constructed in America

The hydraulic fill dam is now recognised as the most economical method of handling earth as well as by far the only absolutely satisfactory means of forming the earth of a dam by alluvial deposit from a current of water, so as to make it more solid impervious and freer from liability to unequal settlement than is possible by any other artificial process Powerful jets of water directed against a cliff or hillside disintegrate the soil, which is carried forward by the same water in rough wooden aqueducts, 2 or 3 feet wide and deep to the site of the proposed dam There the aqueduct bifurcates into two channels which are aligned one along the up stream side, and the other along the down stream side of the dam, and parallel to its axis There the water is discharged from sluices in the sides of the channels Naturally the heaviest detritus is deposited under the aqueducts, whilst the lighter matter

is carried further and the streams from the two channels meet on the central axis of the dam where their deposits consist of the finest impermeable puddle-clay which constitutes the core of the dam Requisite conditions are

(1) The existence of abundance of water at a sufficient elevation to form a sluicing head that is to afford a steep enough slope for the wooden channels which convey the sluice material on to the embankment

(2) An abundant deposit of suitable mixed soil at a high level

The volume of water required is from 5 to 20 cubic feet per second One cubic foot per second will remove 80 to 200 cubic yards in 24 hours The most suitable material is an admixture of soil clay sand and gravel of all sizes Stones of 200 lb weight can be carried through the sluicing channels along with a current of thick clay and sand

In the process of hydraulicking the slopes of coarse material are first formed and are always kept higher than the interior By means of check boards and side flumes the heavier material can always be diverted to the slopes while the finely graduated gravel sand and mud is kept in the centre The open loose stone and gravel deposited outside allows of the drainage and gradual desiccation of the clay puddle which forms the centre portion of the dam

(9) The material is either loosened by a hydraulic jet from a Giant playing against the bank which is the best method or if pressure is not

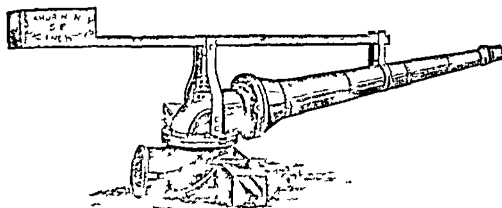


Fig 2 Hydraulic Giant Monitor (Double jointed)

available the ground can be loosened by plough or by hand and then sluiced

Fig 2 is a photograph of a hydraulic monitor The giant is moved round in any desired position by a deflecting nozzle by which device the force of the issuing jet turns the machine on its pivot so that a child could operate it

A long and detailed description of hydraulicking with many photographic illustrations is given in Reservoirs for Irrigation etc by J D

Schuyler an invaluable up to date book which should be consulted on this subject

Wherever earth at a higher level than the dam crest is not available low level borrow pits can be excavated by the jet and pumped up to a commanding level whence they are conveyed on to the dam by flumes

In such cases the outer casing will have to be made of rock fill not by hydraulicking Special arrangements require to be made to drain the internal mass pipes being inserted in the stone casing with the object of removing the water and at the same time retaining the mud in suspension

The cost of hydraulic filling is from $3\frac{1}{2}$ to 6 cents per cubic yard equivalent to $1\frac{3}{4}d$ or $3d$ in English money or from Rs 4 As 6 to Rs 7 As 8 per 1 000 cubic feet in Indian currency

FIG 3 —Necaxa Hydraulic Fill Dam Mexico

(10) A few modern examples of earthen embankments whether made up by ordinary process or by hydraulicking alone or by rock fill or stone walling backed by hydraulic fill will now be given Fig 3 is a section of the Necaxa Dam in Mexico It is one of a series of three constructed for the Mexican Power and Light Company in the State of Pueblo It can store 34 850 acre feet and be used as a penstock for pressure pipes It is thrown across the Necaxa River which has a discharge varying from 70 to 7 000 second feet or cusecs

It was originally intended to build a masonry dam here but the softness of the rock substratum was considered unequal to the concentrated unit pressure of a dam of that description so the hydraulic fill was substituted The material was sluiced down from the adjacent heights where good clay gravel and masses of rock were met with A photograph in Reservoirs of the deposition of rock on the slopes shows pieces of 16 cubic feet which had been sluiced down the flumes This dam is 180 feet high an altitude not long ago deemed impracticable for an earthen dam



Fig 3a —View of construction of Necaxa Dam Mexico

Fig 3a is a photograph of the work during construction derived from the above work

(11) Fig 4 is the section of a dam across the Oigawa River in Japan It is to be of hydraulic fill throughout, except the outermost layer on the slopes which will be rock (presumably hand packed) from the tunnels On the up stream toe is a core wall of concrete with a steel diaphragm built, and the rock filling of the toe is covered with planking, above the point where the steel core intersects the slope, the latter will be covered with asphalt laid over 18 inches of pitching or riprap This dam, when completed, will

Fig 4 —Projected Oigawa River Dam Japan

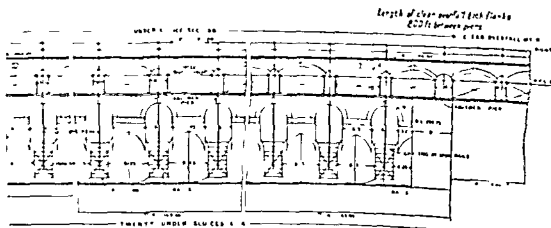
be the highest in the world This is a notable example of the possibilities of and the confidence placed in, the hydraulic fill dam

(12) In Fig 5 is a section of the Maladevi Dam, the most recent construction of that kind in India Both toes are formed of rock *fill*, the up stream one forming a temporary overfall for floods after the first year's work The overfall slope is faced with a thick layer of concrete The whole of the up stream slope is covered with a layer of concrete overlying one of gravel The plans are derived from Strange's "Indian Reservoirs"

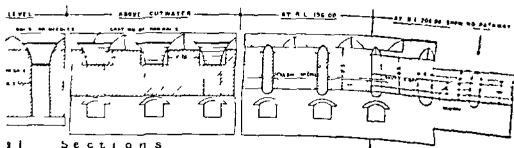
The full and maximum levels are the same This means that the weir cill can be closed or raised by gates, automatic or otherwise The reservoir formed will impound 117,400 acre-feet

The waste weir (Figs 6 and 6a) is of the so-termed stepped type, having three sets of gates at the centre twenty sluices, 6 feet by 4 feet, worked by screws and capstans Next is a battery of eight automatic gates, 8 feet by 10 feet Above all, and extending for its whole length, 1,135 feet of the weir crest is an arcade carrying a footpath and tramway for travelling winches These are provided with 10 × 4 feet teak wood shutters, which are used to temporarily raise the weir crest level by 4 feet The automatic gates it is proposed to use are of Rheinhold s patent, shown in Fig 14, *post* The calculated discharges are as below —

	Second feet.
Free overfall, 800 feet × 4	22,827
Twenty undersluices, 6 × 4	12,000
Eight automatic gates 8 × 10 .	6,457
Total .	41,284 second-feet



Elevation



al Sections

in Maladevi Tank Project

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Section thro (40 1) temporary waste channel

Longitudinal Section shewing gap left 1st Closure

Figs 5 53 —Maladevi Dam Mysore India

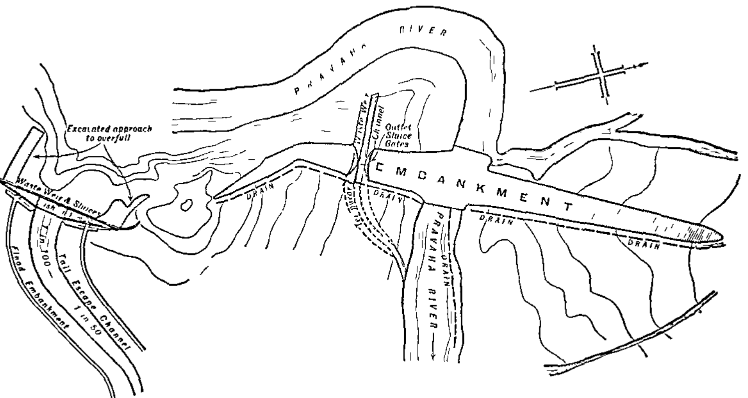


FIG. 5b —Maladevi Dam Site Plan.

This discharge is equivalent to a run off of ·418 inch an hour from the catchment of 153 square miles It is doubtful whether this plan has been

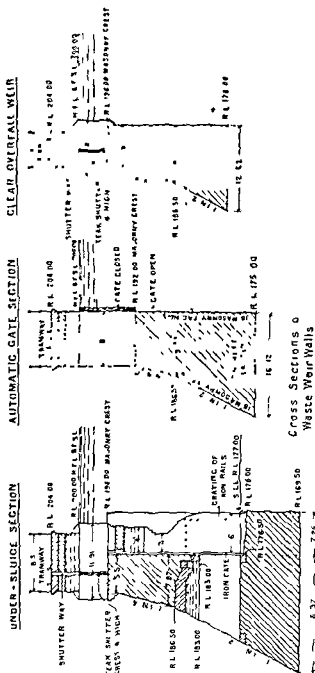


Fig 6a —Sections of Maladevi Stepped ' Weir.

finally adopted, or whether several batteries of the automatic gates used on Lake Fife in the Bombay Presidency have not been substituted This later form of automatic gate has been pronounced a great success The outlets

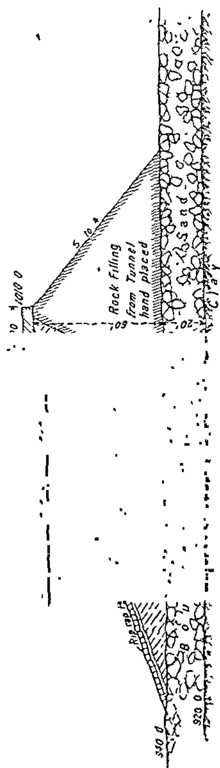


Fig 7 —Zuni Dam, New Mexico.

from this latter reservoir consist of 6 vents 8 feet deep by 4 feet wide, a plan of which is given in Fig. 16, *post*.

(13) The Zuni Dam (Fig. 7a) 720 feet long and 70 feet high is of a different type to the ordinary hydraulic fill earthen dam. It consists of a trapezoidal

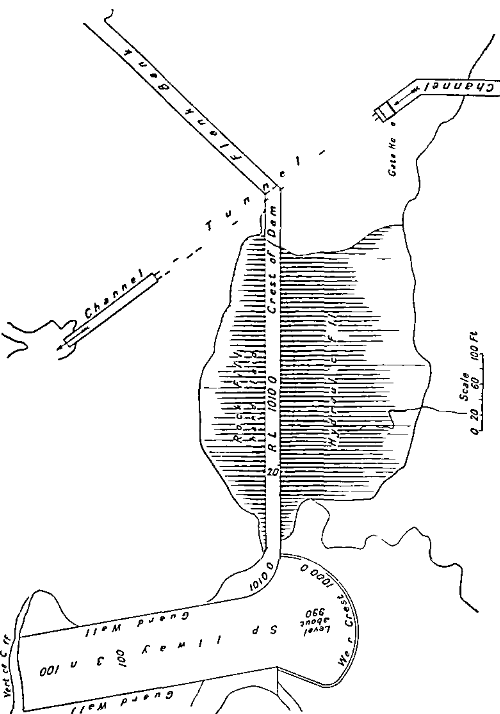


FIG. 7a.—Zuni Dam Site Plan

block of rubble masonry laid dry backed by hydraulic earth fill. The connection with the solid clay substratum through the river bed of boulders

and sand is made by a puddle trench The water for sluicing was delivered through an 8 inch pipe to the hydraulic giant by a steam pump having a capacity of 3 cusecs The work was completed in 1907

The spillway was excavated in rock 100 feet wide and 10 feet deep In September 1909 the dam partially failed through the failure of its spillway the lava rock foundation of which became undermined and subsided 7 feet

(14) An example of pure rock fill dam or rather weir built of dry coursed masonry with a thick skin of concrete on the up stream side and

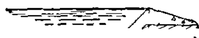


Fig 8—Alfred Dam Maine

the lower 2½ to 3 feet laid in cement mortar is given in Fig 8 of the Alfred Dam in Maine

(15) Fig 9 is of the Milner Dam Twin Falls Canal in Idaho It consists of dumped rock thrown round a wooden plank core which latter is embedded in dry rubble masonry The up stream side is made up of hydraulic earth fill

The wooden diaphragm is for the temporary purpose of preventing the mud of the hydraulic fill washing away through the interstices of the loose rock Once the hydraulic earth fill is drained and consolidated it will become perfectly watertight and the diaphragm can be permitted to decay without any damage accruing

(16) Fig 10 is a section of the Lower Otay Dam in California This dam spans a narrow canyon and it was originally intended to build a masonry weir on the site This accounts for the apparently unnecessary mass of concrete at the base of the dam It is rendered watertight by a diaphragm of steel plate enclosed in a concrete wall 2 feet thick Without acquaintance with all local circumstances it is impossible to judge whether such a dam is the most economical form possible

(17) Fig 11 is a diagram illustrative of the method of rendering a dam formed of sand watertight by means of a puddle core and a puddle trench carried down to solid clay The slopes also are protected by a layer of clay

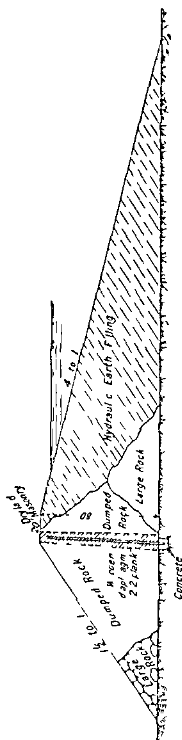


Fig 9 —Milner Dam Twin Falls Canal Idaho

(18) A very instructive example of the capabilities of embankments made of pure sand is given in Fig 12 of the embankment of the Jeypore Waterworks, taken from the 'Min Pro Inst C E,' Vol CXV In this

case the embankment is not only made entirely of sand but also lies on a foundation of sand of great depth The H W L is 44 feet above the floor of the sluice which is at the bed level of the original stream The free board or margin between H W L and crest of bank is greater than usual being 15 feet *i e* one third depth of water and the crest width is 32 feet

The usual dimensions of a clay embankment where a wide roadway is not required would be top width 15 to 20 feet and free board one fourth to one fifth depth of water with a minimum of 3 feet and a maximum of 6 feet and face slope 3 to 1 back slope 2 to 1 whereas in this case the face slope is 4 to 1 The pipe culvert is founded on blocks of brickwork sunk 8 feet below the floor and filled with concrete the inlet tower is founded in like manner The leakage is said to be insignificant the silt deposit on the bed

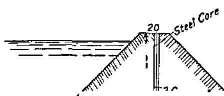


Fig 10—Lower Otay Dam California

of the tank having stanched the sand The value of (l) in this case is 370 feet and the F S L is 30 feet above sill of outlet Consequently $c = 12\ 3$

(19) Fig 13 shows the founding of a puddle wall on concrete which continues the impermeable core down to solid rock In some embankments the puddle trench has to be carried through fissured or inferior rock in this way to even 100 feet in depth

With regard to the thickness of puddle walls the following extract from Waterworks Engineering a recent standard work on the subject will give a safe general rule — For general guidance it may be regarded as a safe rule to make the thickness of the puddle wall at the base of the embankment equal to one third the depth of the impounded water in the reservoir battering both sides at such an angle as will give a common thickness of 6 feet of puddle at the top of the wall This dimension would only apply to a very large embankment It is unnecessary to carry the puddle wall above the highest wave level of the reservoir and it is undesirable for it to

have anywhere a less covering than 3 to 4 feet of ordinary earth to protect it from the co-operative influences of sun and wind The puddle wall must be carried down through the sub oil to form a watertight joint with the impervious base of the reservoir It is laid in a trench excavated in the ground the sides of which frequently battered equally but always reversely to the sides of the puddle wall in the embankment form the requisite lateral support to the material In practice the thickness of this wall at the base is seldom found to be less than half the thickness possessed by it at the ground level The puddle core is sometimes formed of an intimate mixture of sand gravel and clay In the United States peat has been used for the same purpo e but in that country the core is now almost invariably made throughout of rubble masonry or concrete or steel plates enclosed in concrete or a wooden diaphragm is made use of

The Madras rule for the thickness of puddle walls is to make them 2 feet thick at top with a side batter of 1 in 8 or 1 in 6 the base width

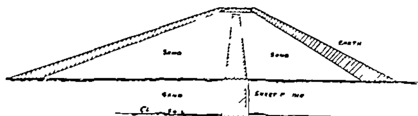


Fig 11

would then become $\frac{d}{4} - 2$ or $\frac{d}{4} + 2$ not very far from the rule quoted in last paragraph of $\frac{d}{3}$ the side batter is however much greater and the top thinner It should be borne in mind however that the earth when carried by coolie labour is far more evenly distributed and much better consolidated than when tipped by trucks in the usual European method hence a thinner section of puddle wall can be adopted

(20) The usual safe side slopes to embankments are 3 to 1 on the water side with 2 to 1 at the rear As regards top width this should not be less than 8 feet in a small tank impounding 10 or 15 feet of water increasing to 15 or 20 feet in a high embankment It is evident that the top width should as in the case of a masonry dam be some function of the depth of M W L and the general rule adopted in Madras that the width of an embankment at M W L should not be less than the maximum depth appears worthy of general use Thus with 50 feet depth and 6 feet free board and 3 to 1 fore and 2 to 1 back slopes the thickness of crest will become 50 (5 × 6) 20 feet But this rule would not be operative for depths much below 40 feet A direct rule for the top width irrespective of the height of free board of $2\sqrt{d} + 2$ would seem to be in agreement with ordinary practice Thus with M W L at 16 feet depth the crest width will be $2 \times 4 + 2 = 10$ feet

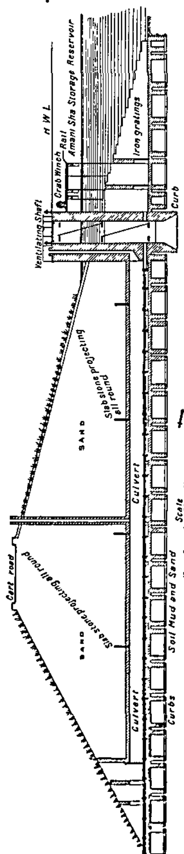


FIG. 12.—Sand Dam, Jeypore Waterworks, India

at 25 feet depth (2 × 5) + 2 = 12 feet at 40 feet depth (2 × 6) + 2 = 14 feet and at 50 feet depth (2 × 7) + 2 = 16 feet

The Madras rule for top width is $\frac{d}{4}$ with a minimum of 8 feet in long tank embankment with varying depths

The height of the free board above M W L is seldom less than 10 feet in large embankments or than 3 feet in small ones the free board is subject to the conditions of wave action

The section of the Waghad Embankment in Fig 1 with a top width of only 6 feet is not considered a good one by Colonel Mullins by whom the group

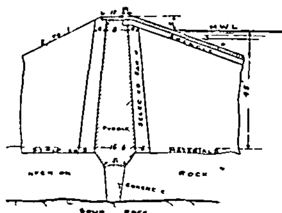


Fig 13

of Bombay tanks are critically reviewed in the Appendix to the Madras Irrigation Manual

The formula for the height of the waves in feet above M W L is $1\ 5\sqrt{F} \times (2\ 5 - \sqrt[4]{F})$ where F is the fetch or longest line of exposure of water surface to wind expressed in statute miles

Thus if $F = 4$ miles the height will be

$$(1\ 5 \times 2) + (2\ 5 - 1\ 4) = 3 + 1\ 1 = 4\ 1 \text{ feet}$$

With $F = 10$ miles the height will be $(1\ 5 \times 3\ 1) + (2\ 5 - 1\ 7) = 5\frac{1}{2}$ feet

Embankments where the soil is unfavourable are often provided with drains longitudinal behind the puddle wall with transverse ones at intervals These are trenches filled with loose stone in order to carry off any leakage and prevent the rear portion being supersaturated thus preventing slips

(21) A masonry dam has this advantage over an earthen embankment in that it can act in whole or in part as a waste weir as for instance the Vyrnwy Weir 60 feet high which has an overflow the Coolgardie Weir 120 feet high and the Bhatgarh overflow weir or else waste or supply sluices can be built in the body of the wall as has been done in the case of the Bhatgarh and Assuan Dams

The design of the section of all kinds of masonry dams and the rules

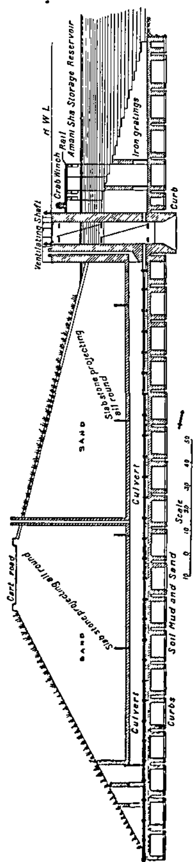


FIG. 12.—Sand Dam, Jeypore Waterworks, India.

(2) *Disposal of Surplus Water*

(22) Where the main dam of a reservoir does not itself act as a waste weir by allowing surplus water to pass over it or through it (if waste sluices are adopted), a separate waste weir or bye wash or spillway has to be constructed. Such is naturally also the case where the dam is of earth. The safe disposal of surplus water passing through an irrigation tank, which is often the receptacle of a large stream, is a most important matter, and the ruin of most old native Indian irrigation tanks is generally due to proper means of escape of surplus water not having been provided. For

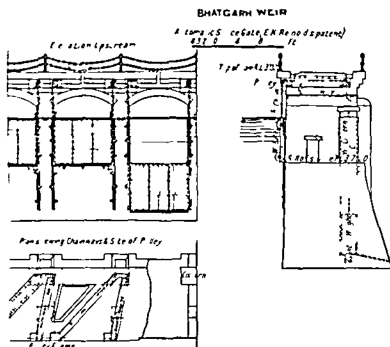


Fig. 14 Bhatgarh Weir Automatic Shutters

designing such a work the maximum possible inflow into the tank under the assumption that the latter is already full has to be ascertained, and the waste weir built of such a length as to be able to discharge this quantity or rate of inflow at a defined depth of film of water passing over the crest. This depth (*d*) is very often 3 or 4 feet, but may be made anything in reason provided that the expense of raising the embankment all through to this extent does not exceed the cost of providing a longer waste weir with a less value of *d*.

It is evident that the crest level of the waste weir is that of full supply level or F S L of the reservoir, while the maximum tank level or M T L (to use the Madras phraseology) will be F S L + *d*. Now the level of the crest of the embankment or dam is naturally dependent on the latter, not the former, so that the adoption of a waste weir as a means of disposal of surplus water involves the raising of the whole embankment to an extent equal to *d*.

governing the shape of the profile have been fully treated in Chaps II and III

The material of which a dam or weir is constructed is usually either rubble masonry or concrete The following remarks on this subject, from Waterworks Engineering (Tudsbery and Brightmore) are well worthy of reproduction —

The construction of the vast masses of masonry or concrete of which dams are formed is an art that requires the exercise of judgment in the selection of the materials used close attention to their preparation and watchfulness during the process of building The question as to whether concrete or rubble masonry is in any given case the preferable building material must largely depend upon the character of the rock available The great tensile strength of Portland cement causes it to be in high favour as a matrix but it is not entirely unexceptionable Where a dam abuts as is frequently the case upon steep hill sides the rapid variation in height tends to produce an unequal settlement in the masonry or concrete as the structure is gradually raised This settlement is resisted by the shear of the material transmitted from the comparatively unyielding rock abutments It is not improbable that action of this kind has produced some of the fractures that have too often occurred in such dams If this be so it points to the prudence of building in a slow setting hydraulic lime thereby permitting free settlement of the mass before it sets rigidly The rapid settlement during construction of large masses of concrete must give rise to internal shearing stress of indeterminate amount In concrete work such settlement is much reduced by the introduction of large blocks of stone and boulders into the work a practice that possesses the advantages of adding to the coherence of the entire work and generally of effecting some economy in its cost

To which remarks might be added that the use of long header stones in rubble masonry laid crossways which is often specified tends to facilitate rupture of the wall With a battering face each face course overlaps the preceding one thus automatically forming a bond between the face work and the interior filling so that long bond stones are not only unnecessary to preserve transverse bond but they have the disadvantage of breaking the longitudinal bond A wall under pressure from the rear has no tendency to split longitudinally as a badly bonded wall of a building might have but owing to unequal settlement the tendency is for transverse cracks to be formed Thus if used at all which is to be deprecated bond stones should be longitudinally not transversely The author considers that the use of special bond stones in rubble masonry is a mistake as tending to destroy the homogeneity of the mass

In the Bhatgarh Dam a section of which is given in Fig 15 Chap II the interior was formed of concrete with a plentiful inter sprinkling of large stones which were embedded in the con rete This raised the specific gravity of the mass considerably above what it would have been without the admixture of heavy stone blocks the actual weight rising to 150 or 160 lbs per cubic foot

The Periyar Dam (Fig 13 Chap II) is also formed of concrete

(2) *Disposal of Surplus Water*

(22) Where the main dam of a reservoir does not itself act as a waste weir by allowing surplus water to pass over it or through it (if waste sluices are adopted), a separate waste weir or bye wash or spillway has to be constructed. Such is naturally also the case where the dam is of earth. The safe disposal of surplus water passing through an irrigation tank, which is often the receptacle of a large stream, is a most important matter, and the ruin of most old native Indian irrigation tanks is generally due to proper means of escape of surplus water not having been provided. For

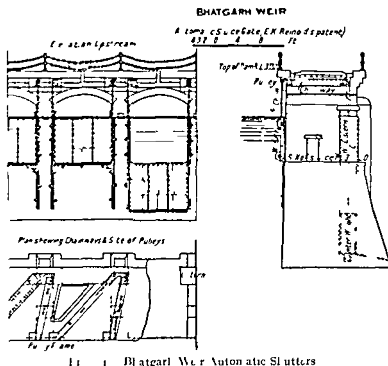


Fig. Bhatgarh Weir Automatic Shutters

designing such a work *the maximum possible inflow into the tank* under the assumption that the latter is already full has to be ascertained, and the waste weir built of such a length as to be able to discharge this quantity or rate of inflow at a defined depth of film of water passing over the crest. This depth (d) is very often 3 or 4 feet, but may be made anything in reason provided that the expense of raising the embankment all through to this extent does not exceed the cost of providing a longer waste weir with a less value of d.

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the greatest allowable depth of film passing the weir This forms the great drawback to the adoption of waste weirs which otherwise are an excellent provision for escapes being self acting

In order to reduce the difference between F S L and M F L to a minimum if not to abolish it altogether several courses are open Firstly The weir could be provided with collapsible shutters balanced at the centre of pressure so as to fall automatically when overtopped They have to be raised by hand which could be effected on the lowering of the water level in the reservoir by men hooking them up into position from a staging erected over the weir This method requires very judicious regula

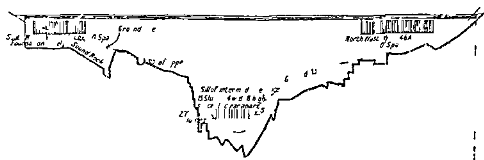


Fig 14a —Longitudinal Section Bhatgarh Dam

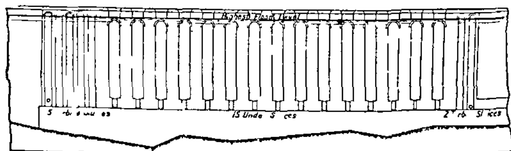


Fig 14b —Elevation of part of Bhatgarh Dam

tion to prevent loss of storage water and is practically limited to depth of 3 or 4 feet Secondly Automatic drop shutters could be adopted as exemplified in the Bhatgarh waste weir (Fig 14) or the Lake Fife waste gates (Fig 16) This system is purely automatic in action the gates being lowered on any increase of level in the water of the reservoir while on a fall of the same below F S L they rise and close the openings Thirdly Sluice bridge openings could be adopted similar to what was suggested for the Kosheshi Escape in Chap X with collapsible upper gates and lower draw gates or only with the former Fourthly An overfall of any kind could be abandoned and the regulation effected by low level sluices built in the body of a closed dam as exemplified in the Bhatgarh Dam and the Assuan Dam (Chap II) Fifthly The cylinder rolling on crest as shown in Fig 29 Sixthly In the stepped weir (Fig 6) where three systems are employed together

(23) The design of the Bhatgarh self acting shutters is illustrated in Fig 14

The principle is that of a counterweight placed in a deep cistern constructed under each pier which counterweight is connected with the gate by a system of chains and pulleys This gate is hung vertically and runs on anti friction rollers An inclined channel through the pier connects an opening which is placed at F S L with the base of the cistern when water rises above F S L it obtains access to the cistern and raises the counterweight and lowers the gate when the level in the reservoir falls below F S L the supply of water to the cistern is cut off and the counterweight falls

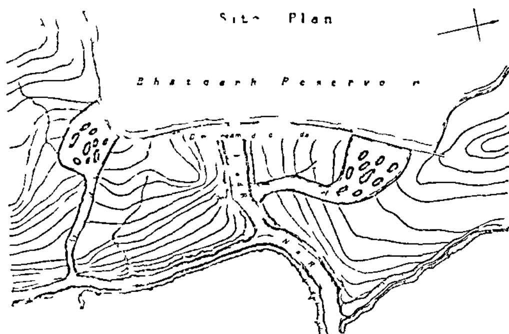


FIG 14c

bringing the gate up into position again *There is a small outlet at the base* of the cistern These gates are said to have given complete satisfaction

(24) The section of the Bhatgarh Dam is given in Fig 15 Chap II This dam is 3 020 feet long and 127 feet high above the lowest point in the foundations It impounds 90 000 acre feet available for irrigation The dam has 15 low level vents 8 feet deep by 6 feet wide and the waste weirs on either flank consist of an arcade with 103 openings of 10 feet Automatic gates illustrated in Fig 14 are fitted to 88 of these the rest being opened and closed by hand The water from the undersluices and overfall weir falls into the bed of the Nira River and further down fills up a low depression termed the Vir Basin which also contributes in a certain degree to the storage capacity At the termination of the Vir Basin is a long waste weir

at the head of which the left bank Nira Canal * takes out It irrigates at present 75 000 acres with a length of main canal of 100 miles and 139 miles of distributaries

Fig 14a is a longitudinal section of the Bhatgarh Dam Fig 14b a part elevation of the deep undersluices and Fig 14c a site plan These sluices have proved effective in keeping down silt deposit and the reservoir in twelve years time has not yet silted up to cill level When it does they are

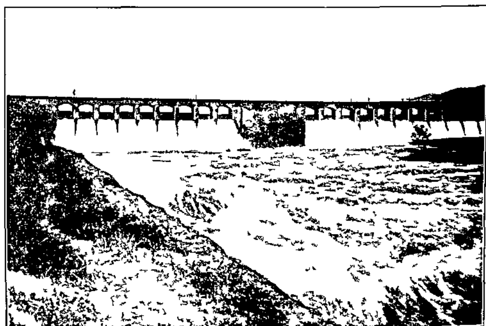


Fig 14d —Single Automatic Waste weir Gates Lake Whiting or Bhatgarh

expected to prevent any further rise in the deposit A photograph of the Bhatgarh gates is given in Fig 14d

(25) The lakes formed by the two dams (see map Fig 15)—the Bhatgarh Dam on Lake Whiting formerly called Lake Bhatgarh and the Lake Fife Dam *alias* the Mutha Dam—are situated only twenty five miles apart, in the Western Ghat Mountains They are built across the Nira and the Mutha Rivers the catchment area of the former of which is but 128 square miles but the rainfall in the catchment varies from 250 inches in the *ghats* themselves to 40 inches at the dam site These rivers discharge immense volumes of water during the actual monsoon months but from January to June practically run dry The object of the reservoirs is to collect a proportion of the flood discharge and store it for the months when irrigation is

mostly in demand The catchment of the Lake Fife i than that of Lake Whiting The Bhatgarh Dam c ,

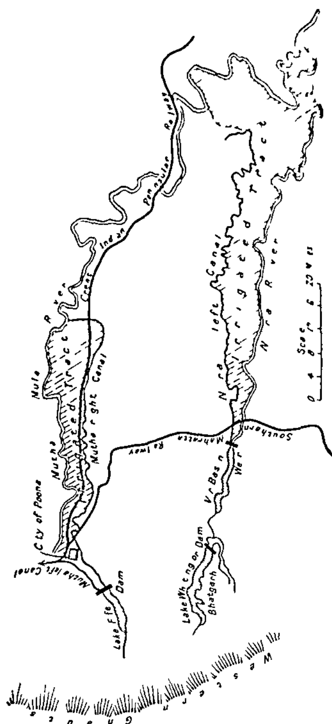


Fig 15 —Location Map of Mutha and Nira Canals

1886 was the first to be provided with undersluices and is consequently the precursor of the Assuan Dam These sluices however do not carry by any

means the whole discharge of the river as is the case with the Assuan but are considered to be mere scouring sluices to keep the reservoir from silting up

The work in the Vir Basin consists of a weir 2 273 feet long and 42 feet high It has two sluice openings closed by needles and two deep under sluices

The Mutha Dam is 3 700 feet long and 98 feet high At its deepest part it impounds 70 000 acre feet available for irrigation

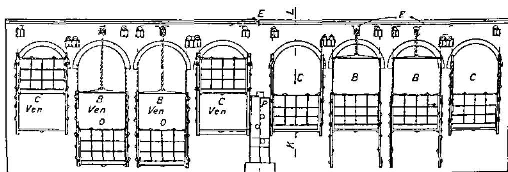


FIG 16 —One Grouped Battery of Automatic Gates Lake Fife (88 gates in all)

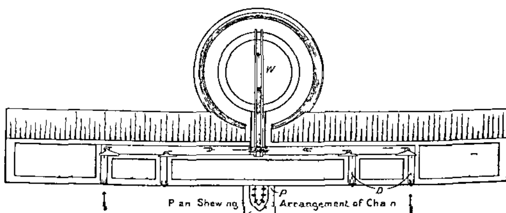


FIG 16a —Plan of Cistern Lake Fife Weir

The map (Fig 15) shows both canals and the areas irrigated by them with their head works

The whole of the irrigation is on sidelong ground The canals cross many drainage lines and consequently have proved very expensive to construct These Bombay irrigation works resemble those in America more than the doab and delta canals in the flat country in Upper India and Madras

(26) In the Mutha or Lake Fife Surplus Escape Weir a new style of automatic closure gates was introduced which has given more satisfaction than the Rheinhold pattern could do as the latter requires a high weir in

which to work properly. The following description is abridged from that given in "The Irrigation Works of India," from which source Fig. 16 is also derived.—The gates work in pairs, suspended from pulleys by chains. The lighter gate is drawn up by chains attached to its ends working inside the grooves, while the heavier gate is fitted with a square hollow frame, the area of which is the same as that of the gate itself. The chain is attached to the centre of this frame, and consequently does not get in the way of the current when the gate is lowered. The heavier gates weigh over 100 cwt., and the lighter one 44 cwt.

The sluices open when the heavier gates are allowed to fall. When fully open the heavier gates lie below the sill of the weir, and the lighter ones rest

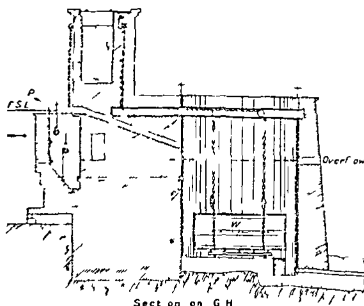


Fig. 16b—Section of Cistern, Lake Fife Weir

above the flood water level. The gates are closed by the action of a counterweight, which is attached by chain or wire rope to the heavier gate. When the heavier gate is lifted the lighter one descends by its own weight. The counterweight works in a round cistern specially constructed for it below the weir. It has a cubic capacity of 760 to 800 cubic feet, but this varies with the depth of the overfall. The lower part below the extreme run of the counterweight is filled with sand.

The plan and section, Figs. 16a and 16b, illustrate the action of the counterweight. When full supply, which is also flood level, is reached, water runs down the channel P into the cistern, gradually immersing the counterweight till at last it floats. The tension in the chains attached to the heavier gates is then relaxed, and the gates fall, opening their own vents, and by pulling up the adjoining lighter gates open their vents also. At the bottom of the cistern is an outlet pipe of smaller size than P. When the flood level falls the supply from P is reduced or else ceases altogether, and

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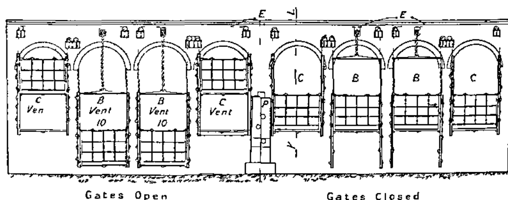


FIG 16 —One Grouped Battery of Automatic Gates Lake Fife (88 gates in all)

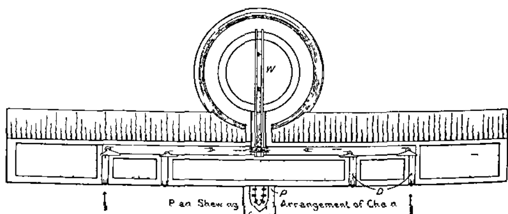


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The sluices open when the heavier gates are allowed to fall When fully open the heavier gates lie below the sill of the weir and the lighter ones rest

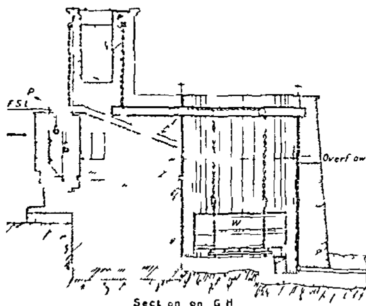


Fig 16b S[illegible] Lake [illegible] Weir

above the flood water level The gates are closed by the action of a coun[illegible] weight which is attached by chain or wire rope to the heavier gate [illegible] the heavier gate is lifted the lighter one descends by its own w[illegible] The counterweight works in a round cistern specially constructed [illegible]t below the weir It has a cubic capacity of 760 to 800 cubic feet [illegible] this varies with the depth of the overfall The lower part below the extreme run of the counterweight is filled with sand

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the water in the cistern then empties itself in a short time through the outlet, thus bringing the counterweight down and reversing the action previously described The outlet as well as the inlet pipes are provided with throttle valves worked by hand when necessary from above There are also some other inlets controlled by valves, by means of which the automatic action can be stopped and the gates held up at any required level

The installation of these eighty eight gates alone cost 5 lakhs of rupees, or £30,000 nearly (150,000 dollars) They can discharge 66,000 second feet

These gates have given perfect satisfaction in working The instalment is undoubtedly very expensive, and would cost more in India than in

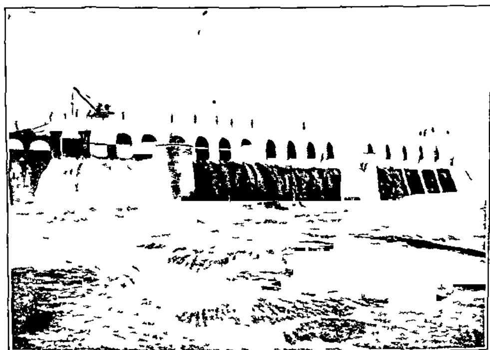
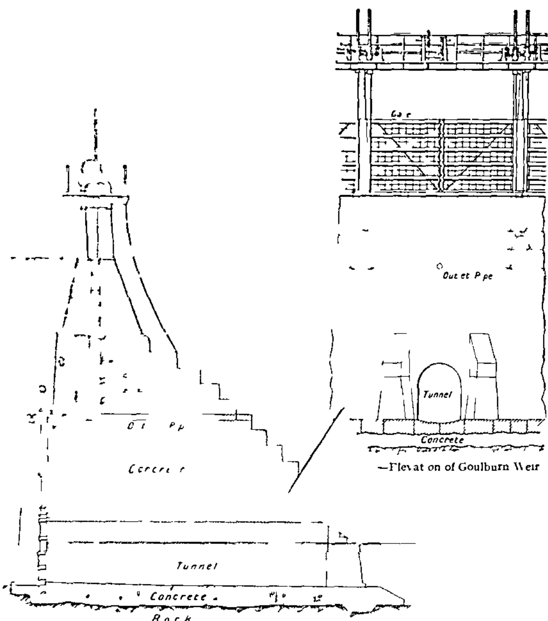


Fig 16c —Lake Fife Grouped Batteries of Automatic Waste weir Gates in action

America, but where the embankment or masonry dam is very long, it saves the cost of the whole work being raised 8 feet, which would be the case if an open overfall weir were adopted, thus effecting a large saving, besides diminishing the risk A view of the gates acting is given in Fig 16c

(27) A section of the Goulburn Weir at the head of the canal system of that name in Australia is given in Figs 17, 17a, 17b This is a drop gate weir Reinforced concrete piers 2 feet wide are erected on the crest 20 feet apart and these spaces are closed by drop gates 20 feet long by 10 feet deep The gates which are manipulated by rack and pinion gearing, are lowered into recesses constructed below the weir crest Owing to the extreme thinness of the piers and the weakening of the section by the gate recess below the weir has to be strengthened by extensive steel reinforcement right down to the level of the base of the recess, and the framework of the

piers is bolted down to this lower reinforcement The view in Fig 17b is derived from ' Irrigation Engineering '



—Elevation of Goulburn Weir

FIG 17 —Section of Goulburn Canal Weir (Australia)

(28) This arrangement which clearly involves considerable expense with no corresponding advantage cannot be commended A better arrangement undoubtedly would be to substitute solid wide masonry piers arched over in the usual Indian style The drop gates can be disposed of by being arranged to slide down the inner face of the weir and when drawn up will run in roller paths up the face of the piers The extra thickness given to the piers will necessitate lengthening the weir but any increase of masonry

in this respect can be economised in the base thickness, which is unnecessarily great

(29) Fig 19 is a section of the Assuan Dam, as originally constructed, through the lowest tier of sluice openings These sluices are 2 metres wide

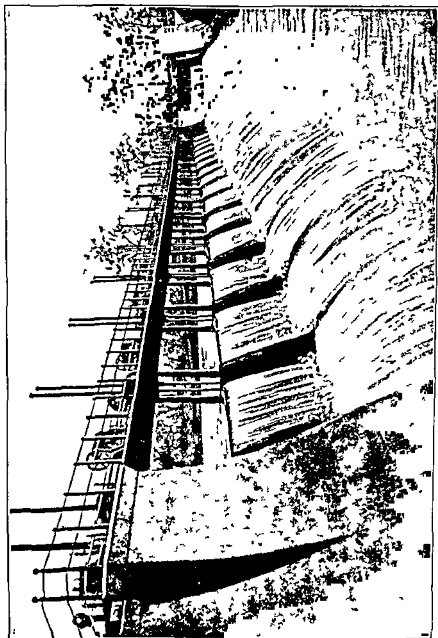


Fig 17b—Battery of Waste weir Gates Goulburn Canal Victoria Australia

and 7 metres high, and the piers are 5 metres in width, with occasional abutment piers

The lifting arrangement with Stoney's anti-friction rollers is illustrated in Chap XIV

An elevation of the dam showing the position of the various tiers of sluices is given in Fig 19b, and a plan over all in Fig 19a This dam, of which a short account was given in par (39), Chap II, has been raised

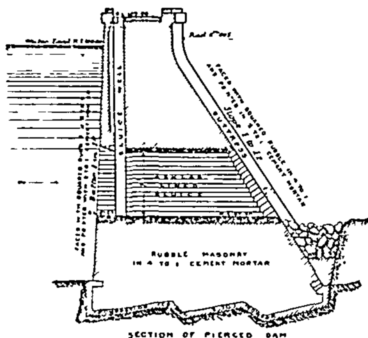


Fig 19 — Assuan Dam (Egypt)

24 feet higher, it will then impound a volume double of what it was originally designed for, and become a really great work

(29a) The original Assuan Dam was designed to store 863,000 acre-feet of water for the irrigation of 420,000 acres of land annually During the

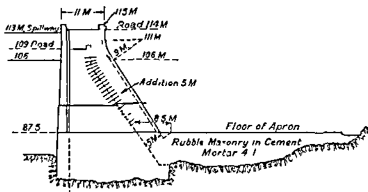


Fig 19c —Assuan Dam Enlarged

period 1907—1912, it was thickened and increased in height by 5 metres (16 4 feet) with a view to increasing storage for the irrigation of 988,000 acres annually The thickening of the dam was effected by adding masonry on

the down stream face to a thickness of 5 metres, measured normal to the face The new masonry was kept 6 inches apart from the old by means of template walls of that depth built on the face 49 feet apart, and the old masonry was tied to the new by steel tie rods, $1\frac{1}{4}$ inches diameter, and $8\frac{1}{4}$ feet long driven into the old masonry at 1 metre intervals in both directions, hori-

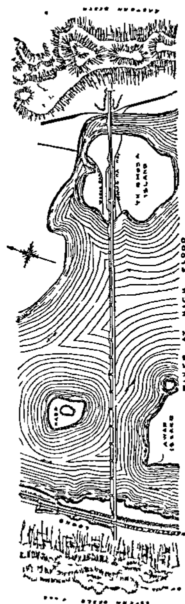


Fig 19a —Site Plan Assuan Dam

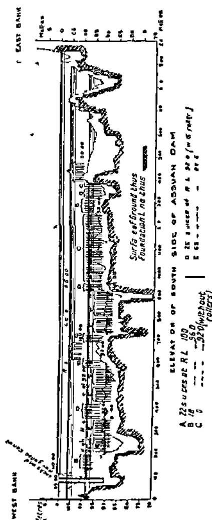


Fig 19b —Elevation Assuan Dam

zontal and vertical The 6-inch space between the old work and the new was then filled up with cement grouting, which was filled in from the bottom of the dam, upwards, through perforated pipes, 3 inches diameter, the pipes being plugged at the top till the time for grouting arrived

Soon after the dam was put into service, it was found that the force of the water issuing from the sluices smashed up the rock of the river bed below

to a depth of 24 feet in places As a protection against this action a masonry apron was laid along the entire length of the dam, down stream (see Fig 19c)

(32) Where local circumstances admit of it, the cutting of a wide spillway or bye-wash will answer the purpose of disposal of surplus water from a reservoir An example of such a work is illustrated in Fig 22 of the spillway

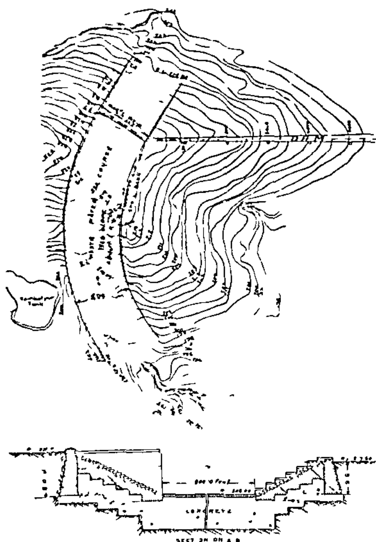


Fig 22—Spillway of Ashti Tank Bombay

of Ashti Reservoir The bye-wash is a cutting 800 feet wide taken through the crest of hill which forms one boundary of the tank embankment, assuming no drop at the head, the slope is, roughly, about 1 in 700, or 1 4 per 1,000 With a 3 feet depth of water passing through, $A = 2{,}400$ square feet, $WP = 806$, $R = 3$ nearly, and Kutter's $N = 0\,025$ $V = 70\sqrt{RS} = 4{\cdot}5$, whence $Q = 2{,}400 \times 4\,5 = 11{,}000$ cubic feet per second If a depth of 5 feet were adopted, the discharge through the bye-wash channel

would be $Q = 4{,}000 \times 78\,4\sqrt{5 \times \,0014} = 26{,}500$ cubic feet. What discharge the channel is intended to carry is not known The discharge per second per *foot run over a weir with free overfall is 17·3 cubic feet when* $d = 3$, *and* 37 3 cubic feet when $d = 5$ The lengths of waste weir required to discharge the same amounts previously arrived at, viz , 11,000 cubic feet and 26,500 cubic feet, would be $\frac{12{,}000}{17\,3} = 694$ in the first case, and $\frac{26{,}500}{37\,3} = 710$ in the second case, the depths of water being the same in either case Thus we see that with this slope if Kutter's $N = \,025$ the discharge is about seven-eighths of that with a free overfall, but if $N = \,0225$ the discharge is the same as that of a free fall

Masonry rapids are a cheaper construction than a series of masonry drops A rapid can usually pass as much water over its crest as if the overfall were vertical

Rapids may be constructed either on a continuous slope, the inclination of which varies with the contour of the ground, or else built in steps Water cushions may be formed in each step by building a narrow wall above the tread at its edge, this holds up water over each step, and serves to dull the velocity of the current A good example of an escape fall is that of the Kushuk Falls (Fig 12, Chap IX) The water cushion here is necessary to check velocity

(33) In the United States many excellent examples of storage reservoirs are to be met with, formed by damming up rivers, mostly in the upper

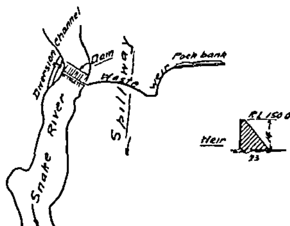


FIG 23 —Minidoka Dam Site Plan

reaches, where rock exists in the flanks as well as in the bed of the river Many such sites are not suitable for canals to take off This is effected lower down *in the course of the river, where another weir or dam of smaller* dimensions, acting only as a diversion weir, is built with the concomitant canal head and possibly scouring sluiceway. The Vir Weir of the Bhatgarh project in India is a weir of this kind In some cases the reservoir is tapped, as was effected in the Periyar project, by a tunnel, the entrance to

which is controlled by sluice gates or by a deep diversion cut in the solid rock at one flank of the weir as in the case of the Minidoka Canal head works. Where canals can take off they often have to run in a winding course cut as a bench in the hill side starting parallel to the course of the river instead of at right angles to it. This arrangement is rendered necessary by the configuration of the ground.

In Fig. 23 we have a section of the dam with a site plan of the head works of the Minidoka Canal the head regulator of the canal of which has already been illustrated and commented on in Chap. VIII. Fig. 21 par. (25). The dam in this case is a combined loose rock and earth dam 60 feet high. The diversion channel whereby the reservoir is tapped is shown cut in the

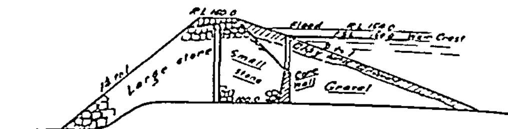


Fig. 23a.—Minidoka Dam Section

rock on the right flank of the weir. This is closed by five gates 6 feet wide and 10 feet high each operated by a pair of geared screws. The sill of these sluices is at R.L. 102.00 or 52 feet below crest of dam.

As is frequently the practice the waste weir is situated on top of one or both the shoulders of the hill adjoining the canyon a spillway being excavated if necessary to a depth sufficient to bring the crest of the waste weir 5 feet or more above the highest point of the spillway. If the waste weir is not raised well above the spillway its free discharge will be greatly hindered by the shallowness of the approach channel. In the present instance no excavation seems to be requisite as from the section given the natural surface appears to be 13 feet below the weir crest and 18 feet below that of the dam. On this the weir is built to a curve on plan following the highest ground line.

(34) A somewhat similar case is illustrated in Fig. 20a Chap. II which is the site plan of the Roosevelt Dam across the Salt River in Arizona.

The dam is curved on plan but it has been designed as though it were a gravity dam not depending on its curvature for stability. A roadway 13 feet wide inside parapets is provided along the crest and continues across the spillways on either flank over two large span bridges 200 feet in length.

These spillways are excavated in the solid rock. They both start at about R.L. 190 and slope up to the weir where the floor is at R.L. 205 and then immediately begin to fall down to R.L. 155 where the rapids merge in the walls of the canyon. The waste water is thus safely conveyed away clear of the dam. These waste weirs are each 200 feet long and 5 feet high

The Salt River project is a very large one, the dam will impound 1,284,000 acre-feet, the area of its watershed is 6,260 square miles, and it is expected to irrigate 200,000 acres annually.

The outlet of this reservoir is made by driving a tunnel 500 feet long, 12 feet wide, 10 feet high, through the solid rock at one side of the dam, in which six gates, each 6·33 feet wide 11·5 feet high, for gateways 5 feet × 10 feet, weighing about 9 tons each and operated by electric motors control the outflow. It is also to act as a silt scouring sluice. It would be a matter of interest to know if these are anti-friction roller gates or whether the old contact * system is still practised, as appears to be mostly the case in the States. The tunnel can discharge 10,000 second feet, which falls into the river channel below. There are several other works on this large project, of which one, the Granite Reef Weir over the Salt River, is illustrated in Chap. II, par. (58).

(35) A further example of a reservoir work is the Folsom Canal Weir, built across the American River in California (Fig. 24). The disposition of

Fig. 24.—Site Plan of Folsom Canal Head Works.

the head works is due to the necessities imposed by the site. The river is crossed at right angles by a masonry weir, which for a length of 180 feet has the crest lowered. This gap can be closed by one long hinged truss gate operated by hydraulic presses, details of which are given in Wilson's "Irriga-

* The gates are not fitted with live roller gear. The greatest pressure on each gate is 800 000 lbs, the force of friction being 320 000 lbs. The gates are operated by hydraulic power, the hydraulic cylinders being operated by electrically actuated pumps. To relieve pressure on gates when they are to be raised a by pass system is arranged for a second set of emergency gates which makes it possible to fill the chamber between the two sets of gates with water under the full head of the reservoir, thus reducing the pressure on the gates to zero. There is a similar arrangement for emptying the gate chamber.—I. D.

tion Engineering The higher portion ends on the west side with a head regulator the canal of which takes off at right angles to the axis of the weir On the east side the weir is turned in a curve of 90 degrees and thus continues parallel to the river up to another canal intake that of the East Canal

The crest of the higher part appears to be only 5 feet above that of the weir crest i e on the same level as the crest of the weir shutters with the evident intention that in case of high flood the long arm of the weir could be overflowed and thus utilised as an emergency outlet for flood water The maximum flood level is said to be 30 feet above weir crest or at R L 225

In both canals sluice gates are provided for the purpose of removing silt while there are three culvert sluices in the body of the weir itself at a low level in order to scour deposit occurring in rear of the weir wall See also Chap II par (61) Fig 34

The Eastern Canal Head is a very massive structure and has been already illustrated in Chap VIII par (27) Fig 23 This head is provided with what are termed sand traps that is silt is deposited in the sand traps and carried away by the sand gate shown at the side.*

The section of the weir is given in Fig 34 Chap II par (61)

(36) Fig 25 is the site plan of the La Grange Weir in California and connected head works A section of this weir has already been given in

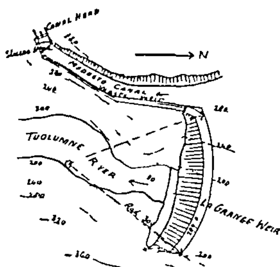


FIG 25 —Site Plan La Grange Weir

Fig 29 Chap II par (57) The arrangement of the canal offtake is similar to that of the east side canal of the Folsom Dam The canal runs parallel to the river on a bench cut in the shoulder of the hill with a flank wall on the left side towards the river This flank wall acts as a waste weir and is

*

practically a continuation of the main weir, but at right angles to it This subsidiary waste weir terminates with a scouring sluice head, close to which, at right angles, is the Modesto Canal Head Regulator This disposition is clearly a better one than that at the Folsom Canal, where the sluice head is placed not close to the canal head, but higher up the approach channel near the weir The clearance of all deposit in the approach channel, which also acts as a sluice way, is thus assured, which is not the case with the Folsom Canal Head On the left of the weir a tunnel is bored through the rock for the supply of the Turlock Canal (not shown on site plan)

(37) Fig 26 is a map showing the course of the Turlock Canal, which is not given in the larger scale site plan of Fig 25

The canal starts with a tunnel through the solid rock, it then falls into

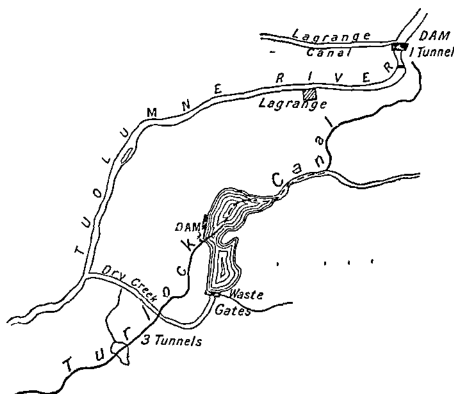


Fig 26 —Site Plan of Turlock Canal

Dry Creek, filling up a large depression which is blocked at a low part of its banks by a dam The canal after leaving this depression which it fills up with water crosses Dry Creek, being here at a higher level, and then passes through a watershed by means of three short tunnels, when it reaches comparatively open country Any excess of water received through the tunnel is got rid of by waste gates or, as they would be termed in Indian irrigation nomenclature, by an escape head which lets it out back into the Tuolumne River by way of Dry Creek

The whole scheme forms an instructive example of the difficulties connected with the alignment of canals in a hilly country

(38) The Dhukwa Weir in Bundelkhund United Provinces of India the profile of which has already been given in Fig 28 par (56) of Chap II is shown again in Fig 27

Like many American example the weir will impound water to a level well raised above the shoulders of the flanks of the river channel proper The work will thus consist of a central overfall weir crossing the defined river channel its continuation on either flank being an insubmergible earthen dam provided with a masonry core wall The two earth dam portions are about 1 000 feet long each and some 35 feet high while the

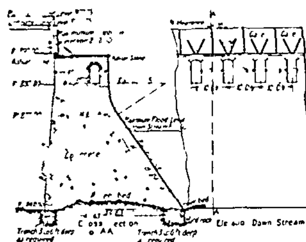


Fig 27 —Section of Dhukwa Weir

central masonry weir is about 60 feet high to crest of the shutters This arrangement is exactly the reverse of that obtaining in the Minidoka Head Works (Fig 23) or of the Roosevelt Dam (Fig 21 Chap II) where the waste weirs are on the flanks the central part being an insubmergible dam

The flood discharge of the Betwa River is estimated at 800 000 second feet and the available impounded supply 86 000 acre feet This reservoir is tapped by three large body sluices 8 × 6½ and 8 × 10 worked by Stoney gates and so as to act as a feeder to the lower old Betwa Reservoir and to the canal which takes off from it How these sluices are to be worked whether from the tunnel or from top of special piers built over each on the weir crest is not stated

(39) The arrangements adopted in this work of dropping and raising the collapsible weir shutters present features of considerable interest as they do not involve the necessity of an over bridge They are fully illustrated in Figs 27 and 27a In the latter figure it will be seen that a rod marked *A*, provided with a hooked end is attached to the base of the movable struts

of each shutter, this end is engaged by the similarly curved end of an arm of the lever *BC*

The lever handles *C* are all situated in recesses connected with a tunnel which runs longitudinally throughout the weir By pressing the handle *C* the rods *A* and *B* are disengaged and the shutter will fall The gates can thus be let go either by an attendant or by the handles being connected with a cable worked from each end of the weir

The raising of the shutters will be effected by means of a crane, running on rails on the weir crest itself The crane as it moves along will be sheltered from the rush of water over the weir by being situated well in rear of the shutters it has already raised in its advance The connection of each strut

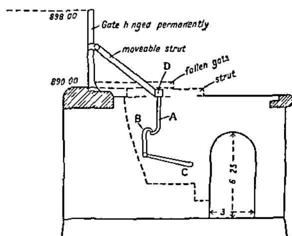


FIG 27a —Suggested Collapsible Shutters

with the lever *BC* will simultaneously be effected by a man operating from the tunnel below

The weir shutters are 8 feet high

The obviously weak points in the design of the regulation apparatus consist in the multiplicity of the gates, which, not being automatically collapsible have to be lowered as well as raised The weir crest is no less than 4 000 feet in length consequently 400 gates will have to be manipulated Arrangements are contemplated whereby they can be lowered in batches, but the raising of the shutters is necessarily a very slow operation In addition to this considerable leakage is bound to occur into the subway by means of the slots through which the vertical arm *A* attached to the longitudinal *D* must move back and forwards as the shutter is raised or lowered These may be covered by stanching strips, but still, under a possible head of 13 feet of water, leakage is bound to occur

Two alternative systems presenting advantages from some points of view have already been illustrated in Fig 34 Chap II, and in Fig 14 In the former that of the Folsam Canal Weir a hinged trussed shutter, 150 feet long and 5 feet deep is raised and lowered by means of a series of hydraulic jacks, the plungers of which act as the supporting struts

(3) *Reservoir and Tank Irrigation Outlets.*

(41) Next to pipes laid through an embankment, which as a rule are only suitable for small tanks upholding 6 feet to 8 feet of water, the simplest

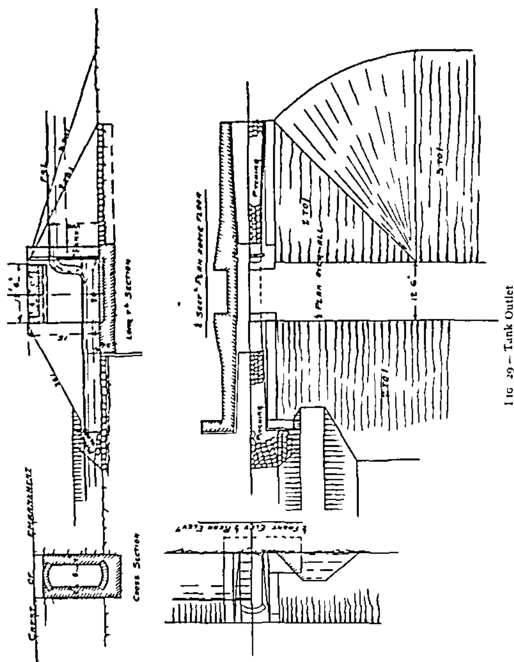


Fig 29 – Tank Outlet

form of outlet is that illustrated in Fig 29 This consists merely of a narrow bridge opening, which is regulated either by double wooden gates either superimposed or moving in double grooves, or by sleepers or baulks of wood, which are slipped into the grooves and removed or replaced one by one as

required The advantage of the latter system is its extreme simplicity and ease of manipulation The water being drawn from the top and having a vertical overfall, the velocity of entry is mostly absorbed On the other hand, when draw-gates are used, the water issues with considerable velocity, the opening acting as a sluice under a head of water The use of double or triple gates obviates this to a considerable extent, as when the top gate is completely withdrawn the conditions become those of an overfall

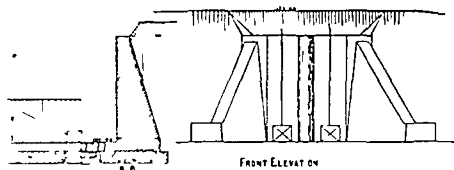
In Fig 29 the opening is 5 feet wide, the floor for the length of the abutments is formed by an invert, a suitable arrangement in this case, as the abutments can be considered as a masonry beam supported at both ends and subjected to a load increasing from the top with the ordinates of the triangle of earth pressure (*vide* Chap I), and so can be designed of light section

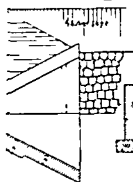
The arch is placed so as to be clear of the water spill at full supply The wings are of the ordinary sloping crested type, and their bases are divergent, 1 in 20 (or the vertical batter multiplied by the slope of crest) from a parallel to the axis of the work On both sides the wings have a crest slope of 2 to 1, that of the embankment on the rear side being reduced gradually from its normal inclination of 3 to 1 to the steeper slope This intermediate portion of bank should be pitched The rear wings slope right down to the ground level which is the most economical arrangement, while the fore wings terminate in short returns so as to form a junction with the bank of the escape channel or canal

(42) When the water is deeper than about 10 feet, this type of outlet will not answer It will be more economical to adopt a barrel culvert for the outlet thus abolishing the abutments The wings, however, will remain, with the addition of a breast wall on the fore side as connection

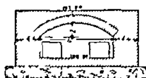
(43) In Fig 31 the sluice openings are double, 3 feet × 2½ feet in size In order to save masonry, the projecting piers and abutments terminate at elevation 117, or 1 foot above maximum rank level, and space for the working platform which is usually of wood, is obtained by sloping back the embankment Access to this will be afforded by a step-ladder Two sets of grooves are provided in order to enable either of the gates to be examined and repaired with a head of water in the tank This can be effected by filling and consolidating earth in the space between the sets of baulks placed in the grooves Figs 31a and 31b are sketches of the gate and its frame The sluice head openings can be lined with cast iron plates or else built of ashlar The roof can be made of rails or rolled beams set in cement concrete, or ashlar slabs could be used Each of these sluice openings, with a maximum free head of 13 feet, will discharge $Ac\sqrt{2gH}$, or $3 \times 2\frac{1}{2} \times 65 \times 8025 \times 36 = 140$ cubic feet nearly, or with both open, 280 cubic feet per second The area of the culvert when full is about 28 square feet, so that the velocity of passage will be 10 feet per second with both sluices open The channel to carry this discharge would have to be of 20 feet base width at least *

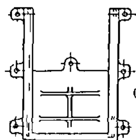
* A better arrangement would be to build a breast wall between the upstream grooves with gate-controlled openings at three different levels each of which would be worked under a head of pressure not exceeding say 4 feet so as to moderate the velocity of efflux —ED

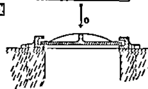


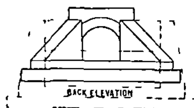












-Double Tank Sluice

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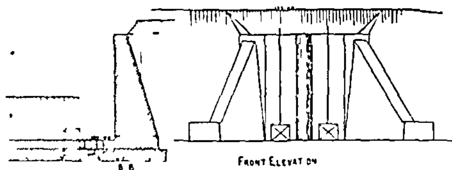
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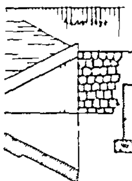
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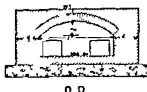
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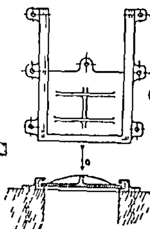
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Double Tank Slu ce

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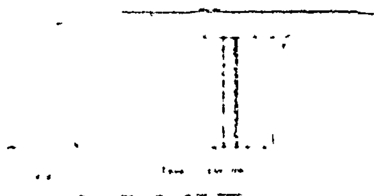
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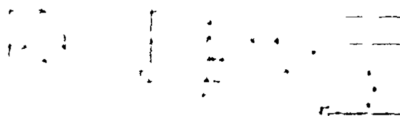
The arch is placed so as to be clear of the water spill at full supply Tl wings are of the ordinary sloping crested type, and their bases are divergei 1 in 20 (or the vertical batter multiplied by the slope of crest) from a paral to the axis of the work On both sides the wings have a crest slope 2 to 1, that of the embankment on the rear side being reduced gradual from its normal inclination of 3 to 1 to the steeper slope This intermedia portion of bank should be pitched The rear wings slope right down to t ground level, which is the most economical arrangement, while the fo wings terminate in short returns so as to form a junction with the bank the escape channel or canal

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(43) In Fig 31 the sluice openings are double, 3 feet × 2½ feet in siz In order to save masonry, the projecting piers and abutments terminate elevation 117 or 1 foot above maximum rank level, and space for tl working platform, which is usually of wood, is obtained by sloping back tl embankment Access to this will be afforded by a step ladder Two se of grooves are provided in order to enable either of the gates to be examin and repaired with a head of water in the tank This can be effected b filling and consolidating earth in the space between the sets of baulks plac in the grooves Figs 31a and 31b are sketches of the gate and its fram The sluice head openings can be lined with cast-iron plates or else built ashlar The roof can be made of rails or rolled beams set in cement concre or ashlar slabs could be used Each of these sluice openings, with a ma mum free head of 13 feet, will discharge $Ac\sqrt{2gH}$, or $3 \times 2\frac{1}{2} \times 65 \times 8$ o × 3 6 = 140 cubic feet nearly, or with both open, 280 cubic feet per secon The area of the culvert when full is about 28 square feet, so that the veloci of passage will be 10 feet per second with both sluices open The chann to carry this discharge would have to be of 20 feet base width at least *

* A better arrangement would be to build a breast wall between the upstream groov with gate-controlled openings at three different levels each of which would be work~ un le head of pressure not exceeding say 4 feet so as to moderate the velocity of efflu





required The advantage of the latter system is its extreme simplicity and ease of manipulation The water being drawn from the top and having vertical overfall, the velocity of entry is mostly absorbed On the oth hand, when draw-gates are used, the water issues with considerable velocity the opening acting as a sluice under a head of water The use of double triple gates obviates this to a considerable extent, as when the top gate completely withdrawn the conditions become those of an overfall

In Fig 29 the opening is 5 feet wide, the floor for the length of abutments is formed by an invert, a suitable arrangement in this case, as abutments can be considered as a masonry beam supported at both ends subjected to a load increasing from the top with the ordinates of the tri of earth pressure (*vide* Chap I), and so can be designed of light secti

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(44) In order to neutralise the velocity of issue a common practice is to build a square chamber in the rear of the culvert with bed sunk well below the level of the floor, and a low weir in front raised above the floor The increased waterway thus given absorbs the velocity and the current issues from the chamber at a moderate pace The outline of this is shown dotted *on the plan and elevation in Fig 31 The banks and bed of the issuing* canal would have to be pitched for a considerable distance below the termination of the floor and wings of the sluice, unless some such remedy were adopted The design was, however, made under the assumption that only one sluice would be opened when supply was at the maximum, the greatest allowable discharge being about 140 cubic feet per second This would give a velocity of 5 feet through the culvert Under such circumstances the well would be unnecessary *

(45) The disadvantages of draw gates to reservoir sluices is that careful regulation is required to prevent waste of water, and the screw lifting apparatus is expensive With deep water the gates require great pressure to force them down The coefficient of friction in rusty gates certainly equals unity This last objection can be obviated by introducing anti-friction rollers in the gates The groove of the frame in this case will have to be made wider and deeper, or else could be abolished altogether In any case, the use of vertical stanching rods will prevent leakage at the sides, and a horizontal one at the top between the upper edge of the gate and the sill By this simple arrangement the frictional resistance would be reduced by 70 or 80 per cent thus enormously relieving the pressure on the screw gearing, which could be made of a very light section in consequence

(46) In the Madras Presidency a system has been successfully employed for many years in smaller tanks, which abolishes all frictional resistance the pressure of water alone having to be met, and that on a very small area, and further enables very accurate regulation of the water to be effected This system consists of admitting water into the culvert by means of circular holes cut in the flat roof of a chamber built in front of the sluice head which holes are closed by conical plugs of wood This is for small tanks of 20 feet depth

Fig 32 contains a design illustrating this construction Before proceeding to discuss this design we will give the following extract from the "Madras Irrigation Manual" relative to this subject

"Taking into account facility, simplicity, accuracy and safety, there are no better means of regulating the discharge of irrigation water from tanks of the ordinary kind than plug holes provided with suitable plugs To secure convenient and fairly accurate regulation it is necessary that (1) the holes should be of a suitable size and sufficient in number, (2) proper plugs should be provided and a well-cut seat made for the shoulder of the plug, (3) the plugs should be regulated from a platform always accessible, (4) it should be known what the discharge is at any given time, and with the tank water

* A better arrangement would be to give the whole barrel a waterway equal to that at *DD*—ED

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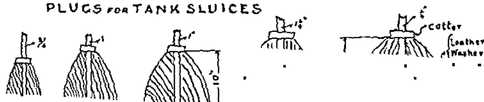
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"Taking into account facility, simplicity, accuracy and safety, there are no better means of regulating the discharge of irrigation water from tanks of the ordinary kind than plug holes provided with suitable plugs. To secure convenient and fairly accurate regulation it is necessary that (1) the holes should be of a suitable size and sufficient in number; (2) proper plugs should be provided and a well cut seat made for the shoulder of the plug; (3) the plugs should be regulated from a platform always accessible; (4) it should be known what the discharge is at any given time and with the tank water

* A better arrangement would be to give the whole barrel a waterway equal to that at *DD*.—Ed.

at any given level, and that the way to secure any required discharge should be readily ascertainable In order to facilitate calculation and comparisons, it is convenient to adopt one ratio of coning for all plugs, and this has been fixed at 1 in 4, also to select the following diameters of plug holes for general use viz, 4, 6, 8, 10 and 12 inches

"It will often be found that a very small expenditure on the improvement of an existing sluice will obviate the necessity for constructing additional sluices, which may be asked for by the cultivators In addition to the plug holes there is usually and properly a low level vent, which is closed and



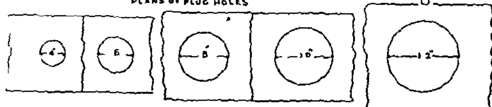


FIG 33 —Plugs for Sluices

regulated by planks or a shutter sliding in grooves The regulation of this lower vent cannot ordinarily be conveniently managed when the water is more than 2 feet deep on the plug stone, corresponding to about 3½ feet on the sill of the sluice It is therefore desirable that the whole area of cultivation under a sluice should be capable of being efficiently supplied through the plug holes alone when the surface of the water is 2 feet above the plug stone

Again, although 2 cubic yards per acre per hour, or 1 cubic foot a second for 66 acres is usually adequate as an average supply, this quantity, when much of the land is being prepared for cultivation, will be by no means enough It is desirable, therefore, that the plug holes should be capable of discharging much more water at times, and this can be provided for by

allowing one or two large or several smaller plug holes so as to admit of the discharge being varied according to circumstances

It is desirable to provide means for raising the plugs to a height just sufficient to secure a full discharge and no more and this will be the case when the bottom of the plug is raised to a distance equal to the diameter of the hole above the plug stone An iron bar (round) passed through the plug and secured thereto by a key or cotter at top and bottom of the plug is the best arrangement for the lower part of the attachment whether a rigid bar be used throughout or a chain be the connection at the upper part with the platform A cross bar with a hole or a collar is the most suitable means of limiting the lift of the plug and a second cross bar should be placed just below the top of the rigid part of the lifting apparatus when a chain is used for the upper part The plug will then be always vertically over the hole to whatever height it may be raised When a rigid bar is used from the plug to the platform the upper part should be flat and should be pierced with holes at short intervals (3 inches will usually be appropriate) as generally with some small and some larger holes in the plug stone the regulation can then be effected with sufficient accuracy Similarly when a chain is used for the upper part of the lifting gear the links may be 3 inches long at the part within the range of the lift

When the plugs are of large diameter and the depth of the water great they cannot be lifted by one or two men and the provision of some sort of mechanical lifting gear will be necessary This may be either a screw or a windlass or an overhead bar with small pulley tackle either of which would enable the regulation to be readily and easily effected A windlass should have a ratchet with pawl or detent to keep the former in any required position

The plugs should be made of hard dense wood and should be turned in a lathe so as to be true to gauge Heart of Tamarind or of Vagay (Tamil) Darasippa (Telugu) makes good plugs They should have iron flash straps at top and bottom of the coning to prevent splitting The shoulders of plugs should be lined with leather the thickness of the layer varying from ½ inch to 1 inch according to the size of the plug and the depth of the water (maximum) This layer of compressible material will diminish leakage and reduce the risk of fracture of the plug stone should the plug be accidentally allowed to drop suddenly

Drawings of different sized plugs are given in Fig 33

(47) The remarks above quoted concerning lifting apparatus require it is considered some modification All plugs small and great should be lifted by screw gear The rods for the larger sized openings under considerable pressure should be hollow of gas or water piping At the end of these rods which should terminate above M W L a brass female screw should be fitted This is engaged by threads cut on a solid round bar near the upper extremity of this latter a circular thrust plate is forged which works in a thrust box fitted with brass bearings above which is the square rod head which takes the handle Thus the power is applied to the male screw which when

revolved enters the hollow pipe drawing it and the attached gate or plug upwards in the same way by reverse action the pipe rod is forced downwards This method of screw gear is far superior to that commonly employed which is to adopt a solid rod threaded at the upper end This screwed end is embraced at the top by a short female screw with a thrust flange and box In this case power is applied to the female screw and the solid rod passes through it rising above This system necessitates a long heavy expensive solid rod and further the screwed end rising above the

Part Up Stream Elevation

Section of Wall

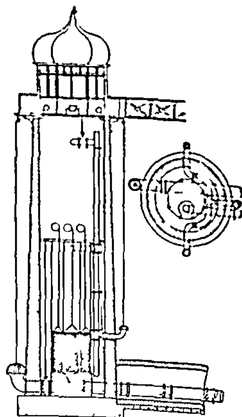
Part Plan over all

Fig 36—Maladevi Outlet Sluices (enlarged)

platform is exposed to the weather whereas in that first described the screwed solid rod is only of a very short length viz that of the vertical play of the gate or plug and can easily be replaced when worn and the transmitting rod being hollow, is well suited to resist torsional as well as compressive strain The solid screwed rod is further protected from the weather and from dust etc as no portion appears above the platform Both these systems are illustrated in Chap XIV Figs 3 and 4 The second system is only suitable for the smaller sized plug holes and should never be adopted for sluice draw gates

(48) Some other points require notice The fore bay chamber is made only 2 feet in height and is shown such in the drawing in Fig 32 This limit

in depth is due to the want of proper grooves and lifting gear the [illegible] arrangement shown in the original plan in the Madras Irrigation Man[illegible] which has been only followed as far as the section of the culvert a[illegible] chamber is exceedingly primitive With proper iron grooves the [illegible] gate even without rollers could easily be lifted by means of an [illegible] wooden windlass under a very much greater mean head than 3 feet wh[illegible] is apparently the limit It is considered that the chamber should be at least 3 feet high to enable it to be inspected except in the case of very small tanks


Fig 37

(50) The Mahadeva Tank outlet (Figs 35 and 36) is a good example of the design of an outlet through a very high embankment the crest of which is 72 feet above sill In such circumstances the usual plan *followed* would be of pipes in a culvert or barrel drain with a valve tower—as shown in Fig 38 In this case however a centre or breast wall has been adopted but with wing cut short the rest of the earth slopes being sloped down in a curve and pitched with stone It is *in fact* a short length of masonry dam interposed in an earthen dam The site was used as a temporary escape during the construction of the dam and was consequently provided with a masonry floor Further the lower part of the breast wall was used as an overfall These temporary works can be utilised in this style of outlet which would not be the case if it consisted of a narrow culvert ending in a valve tower If this were not so *this type of construction would prove more* expensive than a pipe culvert with tower The other works connected with this project are given in Figs 5 5a 5b 6 and 6a of this chapter

(51) The ordinary pattern of English waterworks towers is shown in Fig 37 which is taken from Waterworks Engineering The position of the inlet valves to the tower is adapted to draw the supply from any one of three levels in the reservoir as well as to scour or discharge the water from the bottom The screen arrangement to ward off impurities would naturally not be required in an irrigation work The dimensions of the wall of the tower which is circular are Top thickness 2 feet which increases ½ foot for every 20 feet in depth below maximum surface level of the reservoir

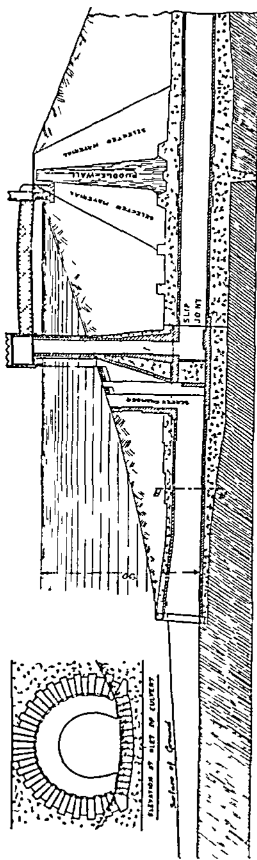


Fig 38.—Hury Reservoir Valve Tower (England)

(52) In order to avoid all earth pressure on the tower, this latter is generally built well clear of the toe of the embankment. If, for economical reasons, it is deemed advisable to build it in the middle of the embankment, this can be done, if the section of the tower is increased to meet the stress thus liable to be brought upon it. The increase in thickness of the tower should naturally be on the far side from the embankment. This is well exemplified in Fig. 38, a section of the Hury Reservoir Embankment, for which we are likewise indebted to "Waterworks Engineering". As will be seen, the culvert is cut in the solid ground, a procedure which should be followed wherever possible.

To allow for uneven settlement in the culvert due to the great pressure a slip joint is provided. This is shown in the section in question. The method of construction of the slip joint is fully explained in the text, to which the reader is referred.

The best section for a culvert to resist pressure is elliptical with the shorter axis horizontal, the ratio is $\frac{\text{hor. axis}}{\text{vert. axis}} = \eta$, the lower third of the section truncated and closed with a flat invert.

The thickness of the arched wall of the culvert is obtained from the formula $t = b\sqrt{\frac{H}{2}}$, in which t = thickness required in inches, b = the smaller diameter of the culvert in feet, H = height of superincumbent earth in feet.

(53) The best method of carrying pipes in a culvert is not to lay them on the floor as shown in Fig. 37 but to suspend them from the roof; this enables inspection to be much more easily effected.

Fig. 39 is a section of the base of the Hubli Waterworks Tower which

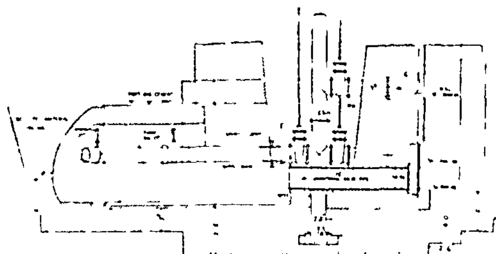


Fig. 39.—Part Section of Hubli Reservoir Tower (India).

shows the pipes hanging from cross T beams built into the sides of the tunnel.

(54) The subject of pipe outlets from small tanks will now be dealt with. These usually consist of earthenware pipes similar to the "Kolaba" pipes used in canal distributaries, with either spigot and faucet joints or else collar connections. In localities where earthenware pipes are difficult or expensive to obtain, excellent pipes can be made of fine cement concrete which is stronger than earthenware and can be moulded into any required shape. In the Ceylon Irrigation Department conical cement pipes are used. These are manufactured on the spot. While on a visit to Anuradhapura some fifteen years ago, the author inspected the manufacture of these pipes, details of which are given in Fig. 40.

As will be seen in the section of bank, the pipes are run up the fore slope of the embankment and are removed and replaced by hand in accordance with the level of the water in the tank and the head required. This is a far superior arrangement to the ordinary pipe head at bed level, the closure of which is commonly effected by a man diving down under water and stuffing a handful of straw into the mouth of the outlet. Another arrangement for regulation is to erect a light wooden open staging over the outlet head. The conical pipes are then fixed vertically between three poles driven round them, and the cultivator standing on the cross bars of the staging can remove or replace the pipes one by one as may be required, leaving the spare ones on the staging or on the adjacent bank.

(55) When a tank embankment dams the course of a large stream which in flood time has to be passed through the reservoir, the question of the

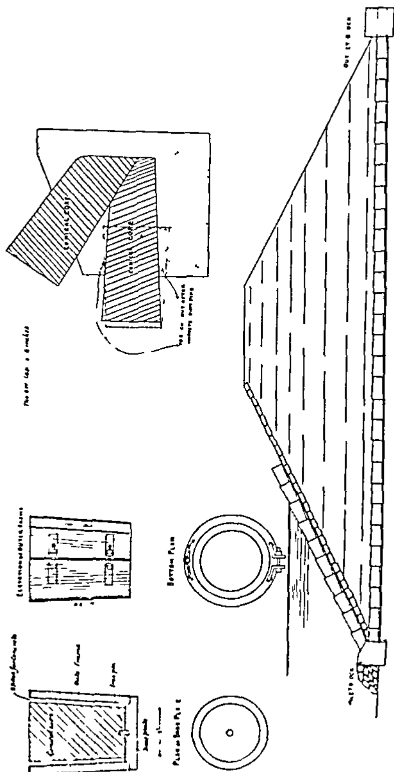
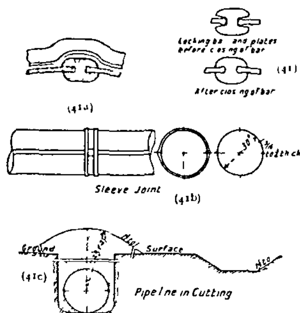


Fig 40 —Ceylon—Concrete Outlet Pipes

possible prevention of silt deposit is a serious one The river risen in flood pours water heavily charged with sediment into the reservoir, which sediment must nearly all deposit within it, thus gradually but surely diminishing its effective capacity Scouring sluices, however powerful, will but palliate the evil, as their action is but local in its effect In such cases the best remedy undoubtedly is to construct some works on the river before it reaches the area of the tank, whereby in heavy floods only the required proportion of the full flood discharge will be allowed to enter the tank the rest being disposed of elsewhere into another drainage line In cases when the whole discharge is required to fill the tank, the current can often, by means of a weir, be forced to spread over an area of flat country on either side, and made thus to deposit its silt on the land before it reaches the depression in which the reservoir is situated

In a large reservoir a floating dredger will successfully keep down silt deposit The possible prevention of silt deposit is a matter which should be carefully thought out on the initiation of any new reservoir project, as many old works have been rendered quite useless by silting up

(56) An interesting paper on the Coolgardie Supply Reservoirs appeared in Vol CLXII of the "Minutes of the Proceedings of the Institute of



Figs 41 41a 41b 41c—Coolgardie Pipe Line (Western Australia)

Civil Engineers" from which the following information regarding a recent work, the largest ever constructed of its kind is deemed of interest —

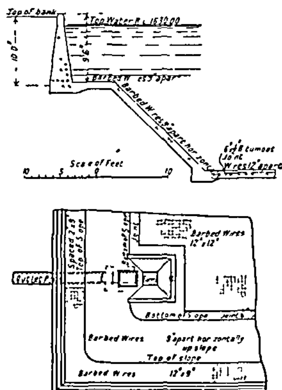
To supply the mines at Coolgardie and Kalgoorlie situated 350 miles from the West Coast an immense reservoir was formed in the rainy belt near Fremantle, on the West Coast having a maximum capacity of 4,600 million gallons The surplus falls over a weir which is the highest ever

constructed Its section is given and discussed in Fig 35, Chap II From this reservoir water has to be pumped, by eight pumping stations, the unprecedented distance of 360 miles

Cast-iron pipes being clearly out of the question, steel pipes 28 feet long and about 2 6 feet diameter were used These are of the locking-bar pattern To quote the paper "The pipe consists of two plates, each the full length of the pipe and each bent to a semicircle The edges are burred or beaded to the shape of a dovetail, and are inserted in the open jaws of heavy longitudinal bars, which are subsequently closed cold on the edges of the plates, thus forming longitudinal dovetail joints The pipes when *in situ* were jointed by a simple sleeve joint, the ring being 8 inches wide *and ½ inch larger diameter than the pipes bulging out to clear the lock bars*"

Fig 41 is a section of the locking bar, open and closed In Fig 41a is a section showing the sleeve joint enlarged, which is leaded, while Fig 41b is an elevation and two sections of the joint and pipe Fig 41c is a section of the pipe way, showing the pipe as covered up by earth

(67) The largest of the service reservoirs, the one at Balla Bulling, is formed of concrete reinforced by barbed wire, and provided with bituminous



Figs 42 42a —Balla Bulling Reservoir

joints right round near the toe of slope This is to allow for expansion and contraction, as no concrete without such relief can stand the alternations of temperature in that climate without cracking

In Figs. 42 and 42a we have a part section and plan of this reservoir. The whole of this immense work cost £2,660,000.

155. In Canada a movable dam has lately been erected over the Red River near Winnipeg, which, although strictly a navigation work, is still of interest to Irrigation engineers as illustrative of a new type. The conditions of the Red River are peculiar: it rises south in the State of South Dakota, and

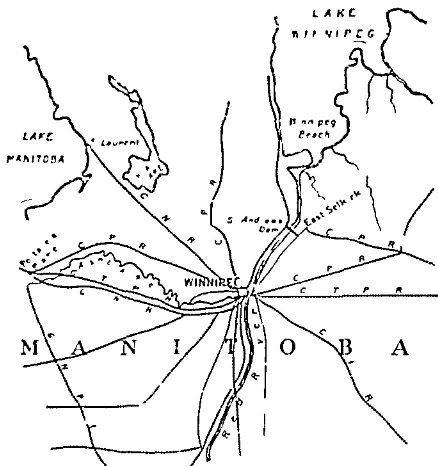


Fig. 43.—Location Map of St. Andrew's Rapid Dam.

consequently comes down in heavy flood in the spring at a time when Lak
Winnipeg and the lower reaches of the river itself are still ice bound. Fo
this reason a movable dam, such as will offer no serious obstruction to th
outflow, is a necessity. The object of the dam is to raise the water in th
Red River to enable steamboats of large size to navigate in the summe
months right up from Lake Winnipeg, which they are at present unable t
do. To effect this the water level at St. Andrew's Rapids has to be raise
about 20 feet, which will then increase the depth at Winnipeg City by 6 fee
The following figures are illustrative of the work.—Fig. 43 is a map showin
the Red River and the location of the dam with regard to the City of Winn

Fig. 43 — Camère" Curtain Dam at St. Andrew's Rapid, Red River, Man.

peg and the lake of the same name. Fig. 44 is a section and elevation of the regulating bridge and movable dam, and Figs. 45 and 46 are photographs of the work, for all of which we are indebted to "Construction," an ably conducted Canadian professional magazine.

The dams of the type known as the Camere Curtain Dam the closure being effected by an articulated wooden curtain which is rolled up and down a frame thus developing or closing the vent It is a French invention

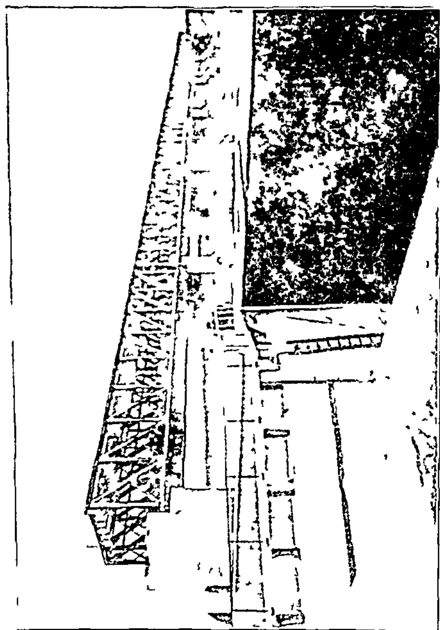


Fig 45.—St Andrews Rapid Dam, curtains up

having been first constructed on the Seine The principle of the movable dam consists in a large span girder bridge from which vertical winged supports carrying the curtain frames are let drop on to a low weir When not required for use these vertical girders are hauled up into a horizontal position below the girder bridge and fastened there In fact the principle is

very much that of a needle dam The river is 800 feet wide and the bridge is of six spans of 138 feet

The bridge is composed of three trusses two of which are free from internal cross bracing and carry tramlines with all the working apparatus

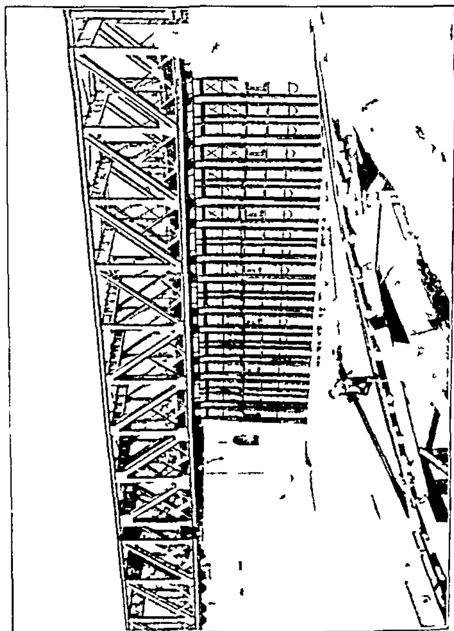


Fig 46 —St. Andrew's Rapid Dam (curtains down)

of several sets of winches and hoists for manipulating the vertical girders and the curtain the third truss is mainly to strengthen the bridge laterally and to carry the hinged ends of the vertical girders

It will be understood that the surface exposed to wind pressure is exceptionally great so that the cross bracing is absolutely essential as is

also the lateral support afforded by a heavy projection of the pier itself above floor level.

In the cross section it will be seen that there is a foot bridge opening in the pier. This foot bridge will carry winches for winding and unwinding the curtains and is formed by projections thrown out at the rear of each group of frames. It will afford through communication by a tramway. The curtains can be detached altogether from the frames and housed in a chamber in the pier clear of the flood line.

The lower part of the works consists in a submerged weir of solid construction which runs right across the river; its crest is 7 feet 6 inches above L.W.L. at R.L. 68[illegible]·50. The top of the curtains to which water is upheld is at R.L. 70[illegible]·0 or 14 feet higher. The dam actually holds up 31 feet of water above bed of river.

(59) The Camere curtain has given satisfaction in France and may possibly do so here. Certain grave disadvantages are, however, inherent in this system. Firstly, the immense surface exposed to wind pressure, which must always be a continual menace to its safety; secondly, the expense of the work, which must be great.

CHAPTER XIII

DESIGN OF CHANNELS

(1) THE method pursued in designing a canal irrespective of its masonry works can only be treated in a general manner. In order to give actual examples of any value for instructive purposes, maps of an irrigation project would be required with a host of statistics, such as the levels of the country affected, the amount of water available at low and high supply in the river, the irrigable area, besides the cost of the headworks necessary to give the required depth of water in the canal, and many other matters. Such information belonging to an actual project not being available, it would be very difficult to produce imaginary conditions whereby an example could be worked out in detail.

(2) The first point to be decided on is the water available in the river of supply; with this information the designer will be able to form an opinion of the maximum and minimum discharge of the canal—regard being also had to the irrigable area commanded, and to financial considerations.

These being settled satisfactorily, the depth of water in the canal is one of the first points to be decided on, as also the sill level of the intake with regard to the weir crest and undersluice floor.

The limiting velocity with full supply will then determine the bed grade of the canal to carry its discharge with the fixed depth, and this slope can be obtained by use of tables.

The bed width of the canal and its side slopes, *i.e.* the sectional area of the waterway, can be simultaneously worked out.

Main canals have to run for many miles from their source of supply without any irrigation branch taking off; how far depends on the slope of the country. As soon as the bed slope of the canal begins sensibly to gain on that of the country, a fall will be required to keep the canal in cutting, and at such sites branches generally take off. As the canal proceeds, allowance must be made by reduction of the base width in each mile for absorption and evaporation, and the same applies to branches and distributaries.

(3) We have seen in Chap. VI. that the question of the quantity and quality of the silt and sand carried in suspension by the water entering the canal forms one of the principal points to be considered in designing an irrigation channel. In most rivers the silt is of a fertilising character, and consequently it is advantageous to adopt means whereby it can be conveyed right on to the fields irrigated, instead of being deposited in the channels near the head.

The inclination given to the bed from point to point should be such that

as it [illegible] the ratio between velocity and water depth [illegible]
the system is any sudden change or checks in the velocity of the current
may cause deposit or erosion. The matters in suspension are composed in
part of relatively heavy sand and in part of relatively light fertile silt.
The heavy sand should if possible be kept out of the canal. The most
effective method to bring this about is to keep the gates of a head regulator
closed during very high floods and to admit only the top water at other
times. This will exclude the heavy sand which is most abundant on the
bed and in the lower layers of the current. The evil can also in some cases
be dealt with by constructing a scouring escape head a short distance below
the canal head. If and when more water is available than is required in
the canal, the escape can be opened so as to pick up and scour out any
deposit which may be on the bed.

(4) The velocity in a main canal should be kept below that which would cause erosion in the bed and banks; the neglect of this important point has led to immense damage and loss, necessitating the eventual regrading of a canal in California, where the first canals were not designed on scientific principles. The following are the maximum velocities allowable:—*

In light sandy soil	1½ to 2 feet per second
sandy loam	2½ "
ordinary firm loam	3 "
stiff clay or gravelly soil	4 "
pitched bed and banks	
shingle and boulders	5 to 6 "

The erosive power of water varies with the volume carried and with the amount of silt in suspension. The maximum velocity adopted should be as high as possible in order to allow a fair velocity when a lower supply is running, otherwise the channels will become choked with water weeds.

(5) The discharge of a canal or distributary is based on the area irrigable and on the *duty** *of water*, *i.e.* the acreage that can be matured by 1 cubic foot per second flowing continuously for a defined time. The duty varies greatly for different crops, seasons and variations in the rainfall.

In Upper India there are two distinct seasons for irrigation when different crops are raised. Firstly the Kharif, that is the summer season from April 1st to September 30th, for tropical crops such as rice, indigo, cane, cotton, etc., and secondly the Rabi or winter season from October 1st to March 31st, when crops common to Europe, such as wheat, barley, peas, etc., are raised. But some crops such as sugar cane require irrigation for eight or ten months, whilst others, such as rice and vegetables, may not need irrigation for more than two or three months.†

* But see Editor's Notes at the end of this Chapter.—Ed.

† In some Provinces the Kharif season is further sub-divided into "Dry Hot Weather," mid March to mid June, and "Monsoon" or Rainy Season, mid June to end of September.—Ed.

The "duty" of the Kharif irrigation varies roughly from 40 to 120 acres, and of the Rabi season from 50 to 150 or even 250 acres * The loss of water between the head of a canal and the fields varies from 30 to 50 per cent

To find the volume of water utilised, the number of days during which the crop is matured is required to be known This is termed the "Base" of the duty The volume utilised will then be found by the following formula —

$V = \frac{B}{D} \times 86{,}400''$, V being in cubic feet (86,400 being the number of seconds in a day of twenty-four hours) Thus, supposing the duty D to be 60 acres, the base B 120 days, then $V = \frac{120}{60} \times 86{,}400 = 172{,}800$ cubic feet of water per acre The depth in feet will be $\frac{V}{43\,560}$, or $\frac{B}{D} \times 1\,51$, or in this case $\frac{120}{60} \times 1\,51 = 3\,2$ feet

This depth can be conveniently expressed in acre-feet,† or symbolically AF, and, being in smaller numbers, appeals to the eye better than the volume given in cubic feet per acre

(6) The use of acres 1 foot deep to represent the volume of water used or stored is most convenient in the case of reservoir or tank storage, the cubic contents of which expressed in acre-feet, instead of millions of cubic feet will then bear a definite ratio to the area of irrigation to be effected Thus if rice which requires 60 inches or 5 feet depth of water to bring to maturity is the crop to be irrigated from a tank, and the effective contents of the reservoir is 10,000 acre feet, then the irrigable area will be $\frac{10000}{5} = 2{,}000$ acres

(7) As already noted canals and distributaries should diminish gradually in bed width to allow for the loss due to percolation and absorption in addition to the reduction in the required water-way due to the taking off of branches or distributaries in the first case, or to that of irrigation channels in the latter case Thus, supposing a canal is estimated to lose 20 per cent of its volume in a run of 100 miles, then, neglecting any loss from water drawn off into branches or distributaries, and assuming the maximum discharge at 3 000 cubic feet per second at the head, the water-way at 100 miles distant should be calculated for a discharge of $3\,000\,(1 - \frac{1}{5}) = 2{,}400$ cubic feet, at 50 miles, $3\,000\,(1 - \frac{1}{10}) = 2\,700$ cubic feet, and at 25 miles, $3\,000\,(1 - \frac{1}{20}) = 2\,850$ cubic feet With the same bed slope the reduction in water-way can be best effected by a corresponding gradual reduction of the bed width

* It varies also, of course, even for the same crop or season or canal, according as it is reckoned on the water supply measured at canal head or at heads of distributaries or other subsidiary channels, owing to loss of water by percolation etc [illegible] —ED

† One cubic foot per second flowing for 24 hours = 86,400 cubic feet, and one acre = 43,560 square feet $\frac{86\,400}{43\,560} = 2$ roughly one cusec or one second-foot flowing for 24 hours = 2 acre feet —ED

(10) To ensure the economical distribution of water the system of *rotation* (or as termed in India *tatil*) has to be *enforced*. That is, that either the whole distributary is closed for a certain fixed number of days in each week, running full supply * the rest of the time, or if the distributary is kept continuously running at the head, certain lengths of it are closed in rotation. This system enables water to be carried down to the end of a distributary and gives to all cultivators a fair share of the water.

(11) In designing canals the discharge capacity will have to be considerably greater than the average discharge obtained from reckoning of duty.

The following example will explain the method of calculating the discharging capacity of a channel under assumed conditions. Area to be supplied 2 000 acres, duty of water 60 acres, then the average discharge of the channel should be $\frac{1}{D}\ \frac{2\,000}{60} = 33$ second feet. But if, as usually happens, the discharge on some days is less than 33, it must on other days be more, otherwise the average discharge will be less than 33. Thus the canal must be designed to a full capacity from 25 to 50 per cent in excess of the "average supply" deduced from $\frac{\text{area}}{\text{duty}}$.

(12) In India the whole subject of the periodical closure of distributaries, or sections of distributaries, and irrigated areas is worked out in advance in a complete manner with great exactitude, so that the system is ready for immediate application as soon as the canals are opened. Very great stress is necessarily laid upon the scientific distribution of water, and from a long course of many years' practical experience this science has been brought to a very high degree of perfection. The financial prospects of a canal system may be said to depend more on the proper distribution of the water available by means of closure tables than on anything else. In India the water rate, which forms the direct revenue of a canal, is charged on the area irrigated, not on the amount of water used; this apparently unscientific system is made to work well and to produce a high duty solely by the application of rules of intermittent supply carefully framed to suit each particular case. The measurement of water itself by modules or weirs, as is practised in Italy and in the States, would be impossible in India, and in any case must involve great difficulty in practical application. In the case of the Calgary Canal, to be referred to later, water will eventually have to be measured out to some 6 000 customers, each legally entitled to a fixed amount; this is an appalling prospect from the point of view of a canal establishment.

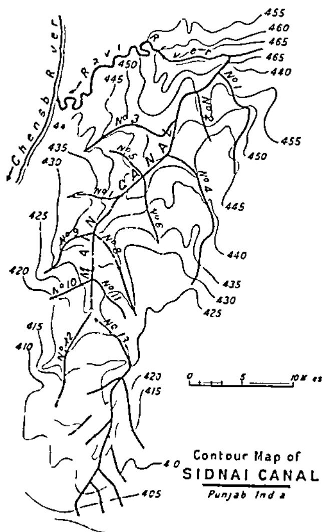
These matters of losses from absorption and of the closure system adopted are fully set out in Mr. Buckley's monumental work, "The Irrigation Works of India," and in Sir Hanbury Brown's "Irrigation," which latter valuable work deals with Egypt as well as India.

* See Editor's Note (T) at end of this Chapter.—Ed.

(13) Two examples will be given illustrative of the alignment of a canal and its branches.

Fig. 1 is a contour map of the Sidnai Canal in the Punjab. This is typical of a good disposition in a flat country. Distributaries as a rule should follow the lines of watershed and thus be free from interference with cross drainage and be enabled to irrigate on both sides. This ideal has been attained in several of the distributaries shown. The general fall of the country is a little over a foot per mile.

The Sidnai is a small canal discharging 2,400 second-feet and commanding 200,000 acres. It takes out of the Ravi River.



Contour Map of
SIDNAI CANAL
Punjab India

Fig. 1

(14) The second example is that of the new canal now being developed in Western Canada by the Canadian Pacific Railway Company, for the irrigation of a large tract in the dry zone east of the Rockies. Reference has already been made to this very important and interesting work in Chap. IX, where three of the falls are illustrated and described; also in Chap. X.

The conditions here are wholly different to those prevailing in the doabs of upper India. The surface is undulating and the slopes are steep. Nearly all the canals and distributaries are consequently in side cutting and are only able to irrigate on one side. If a high ridge occurs, two distributaries have to compass it, one on each side, often not far apart. The heavy slopes add greatly to the expenses of the distribution of water, necessitating frequent small falls in the irrigating ditches, and as the winding contour of the ground has to be hugged, the length of the water way is very much increased above what it would be if a straight alignment were possible; this not only adds to the expense of excavation but must considerably increase the losses of water due to absorption

(15) The land grant belonging to the Canadian Pacific Railway, as shown in Fig. 2, consists of about 3 million acres. Of this area it is proposed to actually irrigate one half, or 1½ million acres. This huge block is divided into three main sections, the western, central, and eastern, of which the western, consisting of [illegible]50 000 acres of irrigable land, has already been brought partly under irrigation. The conditions of land tenure are peculiar. The company sells blocks of land to intending settlers at 25 dollars or £5 an acre, a low enough rate as it includes a water supply right. According to the irrigation laws promulgated in the province of Alberta, land sold as irrigated is legally entitled to a water supply of 1 second foot for every 150 acres, and the company selling the land as irrigated enters into bond to supply this amount of water to the settler. The water supply thus stipulated is in terms of the "Duty" of water. The company undertakes to deliver this, i.e., at a certain commanding point on each quarter section of 160 acres. The supply is therefore net, not gross, or even utilised, i.e., at distributary heads.

In the company's pamphlet it is assumed that a discharge of 2 000 second feet, which is that of the present canal, will irrigate 300 000 acres of the western section, whereas under the conditions stated, allowing for inevitable losses in transit due to absorption, the discharge at the head will have to be 3 000 to 3 500 second feet to effect the aforementioned area of irrigation, and to complete that of the whole project will necessitate a supply from the Bow River of 8 000 to 10 000 second feet, if not more.

At present the land is not thickly settled, but when it becomes so, as is inevitable from the very favourable conditions of soil, climate, and price of land, then the difficulty of supplying fixed quantities of water will begin to be felt, and will present a problem that will tax to the utmost the organising powers of the present able irrigation establishment. In designing the channels no allowance was made for losses in transit, nor for intermittent supply or closure. All these matters, therefore, remain to be worked out, and will necessitate considerable eventual remodelling. It was probably in view of some such contingency that the works constructed are all of a temporary nature, the policy of the company being to get the water through and some start made. It may safely be stated that no Indian administration would ever have undertaken such legal responsibilities as have been imposed on this irrigation company. This obligation, unless considerably modified and relaxed, must eventually result in troublesome litigation, in which the settler has all the advantage.

(16) The Bow River has a minimum discharge of 6 000 second feet, of which it is believed 4 000 have been granted to the company by the Government of Alberta. As we have seen, this will be inadequate even for two sections; consequently the whole low water supply of the river will eventually be required, and any further supply can only be obtained by storage through the formation of reservoirs in some large depressions, which can be filled by the canal during the winter months and drawn upon in the irrigating season. Fortunately, the configuration of the ground lends itself to this arrangement.

The western canal head, or intake from the Bow River, is favourably situated clear of the high bluffs that fringe the river higher up, about a mile

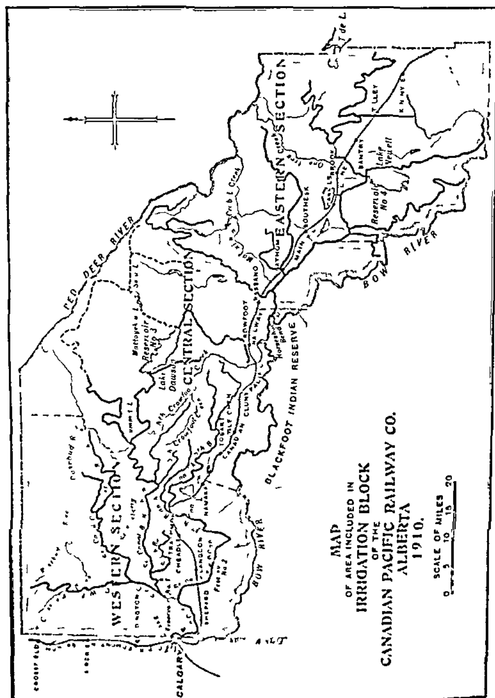


FIG. 2

or more below the city of Calgary (see photo, page 244, Chap VIII) This location suffers, however, from one very serious defect which could only be remedied at considerable expense, and that is its position relative to the city.

However the Bow River is here polluted by the sewage of a large town which pollution unless remedial measures are taken will become worse every year

The result is that the canal water is not potable, and further the remaining stretches of natural water-course through which the canal water passes must likewise be similarly affected

(17) The full depth of the main canal is 10 feet which it retains for its whole length of some 17 miles The bed slope and width have three variations beginning with a 60 foot bed width ending at 44 feet This proportion of depth to bed width is unusually high for a discharge of 2 000 second feet a depth of 8 feet is the ordinarily accepted ratio It will necessitate a weir some 6 or 8 feet in height across the Bow River to bring a full supply which however has not yet been required With a reduced depth of 8 feet in the canal a weir would still be a necessity and the canal would besides have to be widened throughout so that the matter of the most suitable depth is one of comparative cost as some of the cutting in the main canal is of a heavy description

The main canal terminates at Reservoir No 1 from which the so called "Secondary" canals A B and C take out This reservoir is a long natural depression which formation is a common feature in this undulating prairie country and which has been fully taken advantage of in the alignment of the canal The branches *B* and *C* are shown on part of their course on the map (Fig 2) as dotted lines This means that long natural depressions are made use of Some of these which are very deep can be embanked and used as reservoirs to be drawn upon as occasion demands and they are of great advantage for watering stock Reservoir No 1 is 40 feet deep in places, only a few feet at the surface can however in this case be drawn upon as storage

The secondary canals are of about 30 feet bed width and 8 feet deep at their heads and are altogether 150 miles long The distributing ditches will total 800 miles

The secondary canals correspond more or less with large Indian *rajbahas*, or distributaries the ditches with village "water-courses

The allowed mean velocity in the branches averages 2 3 to 2 7 feet per second the soil being very friable this is about the limit which could be adopted without erosion The side slopes are 2 to 1 and unusually flat inclination Most Indian canals are dug out 1 to 1 and eventually the side slopes are supposed to silt up to ½ to 1 Fig 3 is a view of the heavy side cutting near the canal head at Calgary

(18) The western section already effects a considerable amount of irrigation The soil differs from the black soil of the so called prehistoric Lake Agassiz which covers the greater parts of the provinces of Saskatchewan and Manitoba in colour and texture It is lighter and has not the great depth peculiar to the more eastern provinces the soil in which is nothing more or less than a variety of the familiar black cotton soil of India sticky when wet hard and split up when dry The Alberta soil is however, very fertile and only needs water to raise the most astonishing out turn of wheat sugar beet and other valuable crops



FIG 3 —Side Cutting, first 2 miles Calgary Canal Looking down stream

The canal extension in the eastern section has lately been put in hand. This portion will be supplied by a new intake from the Bow River, situated at Horse Shoe Bend, near the railway station of Bassano

The river will be dammed by a masonry or concrete weir 40 feet in height which is believed to be under construction The settlement of this section is now being proceeded with This reclamation closely resembles that so successfully brought about in the Lower Chenab Canal in India The central section will be supplied from the existing Calgary intake assisted by storage in natural reservoirs the principal of which is the picturesquely termed Dead Horse Lake recently altered to Dawson Lake

These reservoirs can be filled in the non-irrigating season and drawn upon later This system of intermediate storage is a novel and most interesting feature and its successful working out should be watched with interest by the profession

The main canal and some of its branches will have to be widened to carry the additional supply of the central section probably to the eventual total

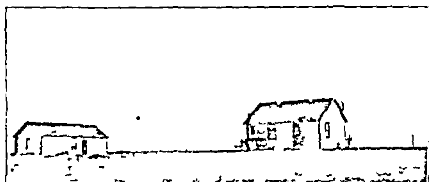


Fig 4 A Canal Choki on the prairies

extent of 5,000 second feet The author had the privilege of being taken round a portion of the western section by Mr H B Muckleston Chief Division Engineer who from having mainly to do with the design of these complicated works was well qualified to afford information on technical points This officer s first experience in velocity observations of a practical nature was in the Johnstown reservoir disaster from the flood waters of which, like Moses of old he was rescued when a child The Johnstown (Pennsylvania) flood brought about by the bursting of a reservoir is notorious as the most disastrous on record Fig 4 is that of an Inspection House in the western section and will be interesting to Indian irrigation officers who spend a large portion of their official lives in canal chokis

(19) Canals in the North West have only one irrigating season as in winter ice and snow cover the country the winters in Southern Alberta are however much milder than in Manitoba and Saskatchewan owing to the proximity of the Pacific Ocean and the warm Japan Gulf Stream which laves the western coast

The canals can therefore be used for water carriage to reservoirs during the best part of the winter

In British Columbia considerable orchard irrigation is effected but the canals are on a very small scale Mountain streams are intercepted and

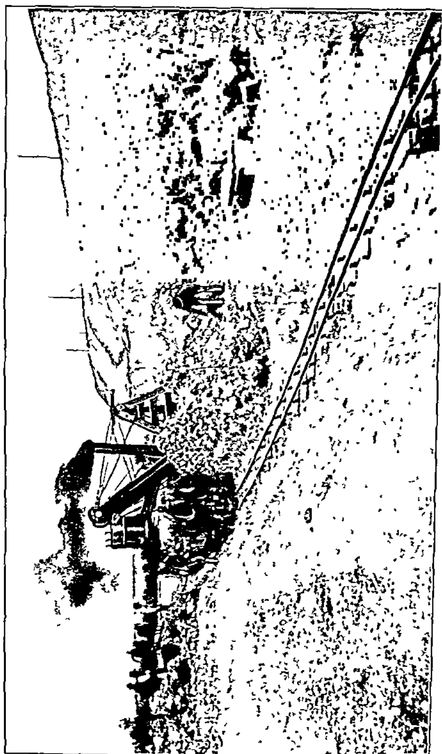


FIG 5.—Excavation of side cutting near head of Calgary Canal.

conveyed by ditches dug along a falling contour in the hill-sides to the orchards and fields Irrigated land here sells for 200 or 300 dollars, *i.e.*, £40 to £60 an acre, so that irrigation is an exceedingly profitable undertaking.

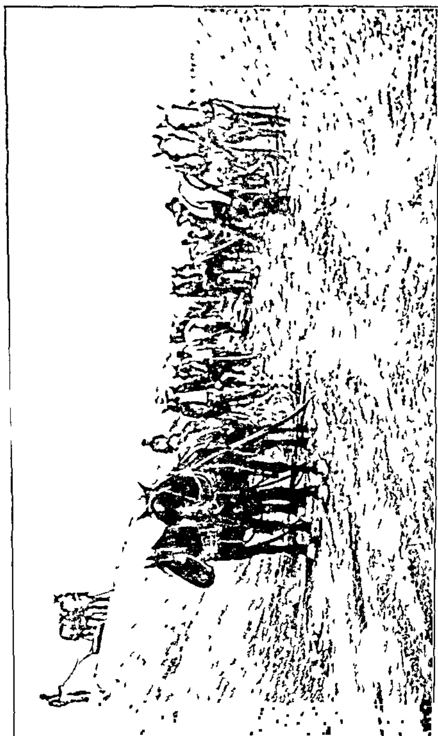


Fig 6 —Horse Scrapers at Work Calgary Canal

Water is however, deficient, and storage works will soon become an urgent necessity

The valleys with large lakes are occupied and cultivated but the high

plateaux on top of the adjacent hills are at present a waste except as a cattle range, being almost impenetrable owing to fallen timber. Here large lakes of water exist which could be made to form reservoirs for irrigation and mining purposes.

Some rivers, such as the Similkameen, which flows into Washington State, present tempting prospects of irrigation, and their wide deep beds could be transformed into a series of large reservoirs. Unfortunately the Northern Pacific, an American railroad, has been allowed to run their Vancouver approach line up this particular valley, thus effectively hindering any such future utilisation of its flood waters.

(20) The two illustrations, 5 and 6, depict Western methods of excavation which are of interest when contrasted with Asiatic practice in effecting the same purpose.

EDITOR'S NOTES

(S) *Mean Velocities—Silt and Scour* (par. (4)).—It has long since been recognised that the erosive and the silt-transporting power of a current of water depends, not on its absolute velocity, but on its *relative* velocity, relatively, that is to say, to the depth and width of the stream. In the year 1867 Mr. T. Login, an executive engineer of the Upper Ganges Canal, as the result of observations made on that canal during the period 1856–63, expressed opinion in a Paper read to the Institute of Civil Engineers (Vol. XXVII.), "That the power of water to hold matter in suspension varies directly as the velocity and *inversely* as the depth." This came to be known in Europe and America as "the Indian theory." In 1863, in his treatise "Etudes Théoriques et Pratiques sur les mouvements des Eaux Courantes," M. Dupuit showed that, if sand or gravel be placed in water in a vase which is caused to rotate, the heavier bodies are carried up in the water, in pro-
p[illegible] e theory
th[illegible] ifference
of [illegible] to right
and left of a solid body, engenders a force which pushes it to the side of the more rapid filaments. The relative velocity of the filaments in the vertical direction will engender a pressure which will carry it to the side of the more rapid, *i.e.*, it will act from below to above."

In 1894, as the result of experimental observations of flow in channels of the Upper Bari Doab Canal, Mr. R. G. Kennedy, a Punjab irrigation engineer, presented to the Institute of Civil Engineers a Paper containing an empirical formula as a guide to the design of irrigation canals, with a view to the prevention of silt deposits or erosion of bed. This formula was $V_0 = cd^m = 0.84d^{0.64}$, wherein d represents the water-depth of the stream, and V_0 the "critical" mean velocity, which is the minimum necessary to prevent silt deposit, and the maximum consistent with freedom from erosion in soil of average firmness.

Since the year 1900, or thereabouts, this formular has met with considerable acceptance in India, and even outside of India, though it has been considered necessary to modify the numerical values of the constant and of the index in different localities. The engineers of Sind say that their critical velocity is about 0.75 of Kennedy's V_0; the engineers of an irrigation canal offtaking from the Colorado River at Yuma (Cal., U.S.A.) modify the

cill of Canal Head = 450 feet Then discharge per foot-run $= \frac{10{,}000}{450} = 22.2$.

Let the cill be at or above canal water-surface, so that the entry is by 'free' overfall Then $3.33h^{\frac{3}{2}} = 22.2$, and the depth of water over cill

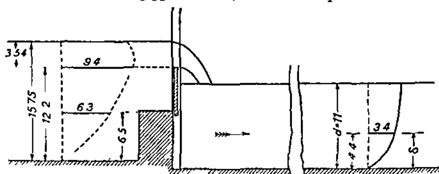


FIG 8a FIG 8b

will be $h = 3.54$ feet The mean velocity of entry, $v_r = \frac{22.2}{3.54} = 6.27$ (say, 6.3) and maximum velocity, at cill level, $v_{mr} = 6.27 \times 1.5 = 9.4$.

Let δ_{mr} be the elevation of cill above river bed, *i e* above the level down to which bed can be kept scoured by weir-sluices Then, from $\frac{v_{mr}}{\delta_{mr}} = 0.77 = \frac{9.4}{\delta_{mr}}$, we deduce $\delta_{mr} = 12.2$ feet This gives the appropriate level of the cill Instead, however, of building the crest-wall up to that height, it may be built up to 6.5 feet only, but, furnished with a movable gate cill, sliding in a groove behind it, up to the required level (see Fig 8a) It is only in the high-water (or flood) season that the cill is needed as a silt-excluder In the low-water season the gate-cill may be sunk behind the masonry crest-wall

Or, the masonry crest-wall may be built 8 feet high, in which case the cill gate, also 8 feet high, can rise to 15.75 feet above river bed, and, by abutting against a lintel at that level, close the canal altogether

In the foregoing, the Silt-Index of canal head has been based on the *maximum* velocity of entry, *at cill level* If we had based it on the *mean* velocity, whose level would be $\frac{1}{3}$ *h above cill*, the equation would have been $\frac{v_r}{\delta_r} = 0.77 = \frac{6.27}{\delta_r}$ Whence $\delta_r = 8.1$ = height of cill.

Working on the maximum velocity, we may make the cill 12·2 feet high, instead of 8.1 feet only, by way of factor of safety against the more violent nature of the eddies—both horizontal and vertical—set up by the cill, and cut water piers, if the waterways of the canal head be relatively narrow and conducive to eddying

In the foregoing illustration we have designed the gate cill to discharge by "free" overfall, for the sake of simplicity of calculation That condition is not, however, absolutely essential The gate-cill may be submerged below tail-water, and yet serve efficiently for cill-exclusion, so long as $\frac{v_r}{\delta_r}$ or $\frac{v_{mr}}{\delta_{mr}}$ does not exceed $\frac{v}{\delta}$

The Silt Index being merely representative of the comparative force of the silt-suspending eddies at two places only a few hundred yards apart, it matters not what the coarseness of the silt may be, whether sand or shingle, pebbles or boulders, so long as I is kept lower than I the water entering the canal [illegible] will not bring with itself detritus too heavy for the currents of the open channel to carry on.

This Silt Index rule of design is consistent with the recorded hydraulic conditions of Punjab perennial canals, as set forth in a paper on "The Design of Canal Head Works" read at the Punjab Engineering Congress of 1914.

(I) *[illegible] Losses.* At the Punjab Engineering Congress of 1917 at Lahore Mr E W Woods suggested that losses by absorption including evaporation from water channels in earth might be calculated by the formula $Q = c\,a\,d$, where Q represents the loss in cusecs, c a coefficient varying inversely with the hydraulic mean depth, a area of reduced wetted surface of channel (bed area *plus* half area of side slopes) in units of 1 000 000 square feet, d water depth on bed (average).

Evaporation losses are only about 6 per cent of percolation losses. When side slopes are ½ : 1 the reduced wetted surface = water-surface nearly.

The following are some values of c

TABLE I

H[illegible] mean depth	[illegible]	n Q = [illegible] a d	Hydraulic mean depth (r)	Reciprocal 1/r	[illegible] in Q = c a d
1	0 1000	1 157	5	0 2000	1 637
9	0 1111	1 220	4	0 2500	1 830
8	0 1250	1 294	3	0 3333	2 113
7	0 1429	1 383	2	0 5000	2 588
6	0 1667	1 494	1	1 0000	3 660

The Table below shows losses calculated by this formula for a canal with bed width 200 feet, slope 1 in 6 250, side slopes ½ : 1, Kutter's $N = 0.025$ —

TABLE II

Discharge (cusecs)	Mean (V)	Water depth (d)	Hydraulic mean depth (r)	( ) in Q = c a d	Q = Loss by absorption (cusecs)		
					Per million sq feet of channel surface	Per mile (5 000)	Per cent per mile
4 900	2 99	8 0	7 5	1 337	10 7	11 2	0 23
2 972	2 44	6 0	5 7	1 537	9 2	9 5	0 32
2 238	2 21	5 0	4 8	1 672	8 36	8 59	0 385
1 908	2 05	4 6	4 43	1 740	8 0	8 21	0 43
1 519	1 88	4 0	3 87	1 855	7 42	7 6	0 50
613	1 27	2 4	2 35	2 382	5 72	5 8	0 95
454	1 13	2 0	1 96	2 610	5 22	5 28	1 16

The loss per unit area of surface varies directly as the water-depth, but the percentage of discharge lost varies inversely with the water depth Hence the advantage of working canals under full supply conditions as far as possible

In the Punjab a record is maintained of the daily water supplies passed into each canal at its head, and also a similar account for each distributary head The difference between supply at canal head and the aggregate supplies delivered to distributaries represents the volume lost by absorption in the canal The table below shows the absorption calculated by formula, as compared with the losses thus recorded statistically during the twelve-year period 1899—1912, for the principal perennial canals of the Punjab, the unit discharges recorded being "day-cusecs," or cusecs flowing for twenty four hours

TABLE III

Canal	Full capacity (cusecs)	Kharif season			Canal mileage	Absorption losses			
		Statistical record		Loss by formula (cusecs)		Per mile		Per cent of discharge	
		Average flow (cusecs)	Loss by absorption (cusecs)			By statistical record	By formula	By statistics	By formula
Upper Bari Doab	6 500	5 046	895	893	340	2 63	2 63	17 7	17 7
Sirhind ,	8,000	5,175	881	913	482	1 83	1 90	17 0	17 7
Lower Chenab	10 730	8,822	1 497	1,622	428	3 50	3 79	17 0	18 4
Totals	25 230	19 043	3 273	3 428	1,250	2 62	2 74	17 2	18 0

From which we may infer that the absorption losses of canals of those magnitudes are about 17 5 per cent of average flow at canal head, and about 2 7 cusecs per mile of canal

(U) *Duty" and "Full Supply Factor"*—More misconception is prevalent on this subject than on any other question of irrigation "Duty" stands to "F S Factor" in the inverse ratio of "average demand" to "peak demand" "Duty" is the area irrigated by a canal, in acres, in an understood period of time, divided by the "average supply" of the canal in cusecs in that time "F S Factor," or "full capacity duty," is the same area, divided by the full capacity in cusecs of the canal The period of time understood is the *Kharif* season, from April 1st to September 30th, or the *Rabi* season, from October 1st to March 31st If any other period be reckoned with, it should be specified, otherwise the "duty" will be either meaningless or misleading The volumetric record of canal flow is maintained in terms of the "day cusec" as unit, though the prefix "day," being understood, is usually omitted in common parlance Fortunately the day cusec (86,400 cubic feet) is roughly equal to 2 acre-feet (43 560 × 2), and the volumetric supply in acre-feet per acre irrigated, can be conveniently expressed by the Greek letter "Δ"

The " du [illegible] is measured at canal-head, or at [illegible] at head of the private watercourse [illegible] the loss of water by "absorption" *en route*

In the summer there is usually plenty of water available for irrigation canals, yet, owing to the vicissitudes of agriculture they flow to full capacity only for brief periods of peak demand In the Punjab their average Kharif flow is usually about 80 per cent of full capacity. The kharif rajbaha-head "duty" is usually 100 acres per cusec of average supply Thus a rajbaha of say 150 cusecs capacity will have an average kharif supply of 120 cusecs, and, with that should irrigate 12 000 acres And its "Kharif F S Factor" will be 12,000/150 = 80 In designing new channels the F S Factor should be reckoned with, in order to arrive at the "full capacity" of design, but it is a common mistake to confuse "F S F" with "Duty" This mistake appears to have been made in the design of the "Rice Canals" of the Sukkur Barrage Irrigation Project, of Sind, now under execution Since about 20 per cent of canal head supply is lost by absorption between canal head and rajbaha heads, 25 per cent must be added to the aggregate full capacities of rajbahas, in order to arrive at full capacity at canal-head

The duration of irrigation of a rice crop may be two or three months, or even five months In case of such crops the term "duty" should not be used without specification of period, the water account should be kept in terms of "acre-feet per acre irrigated," or of "Δ"

The "duty" of irrigation water in the *rabi* season is approximately double of that in the *Kharif* season, excluding rice irrigation The ratios of Δ usually are —Rabi Kharif (barring rice) Rice 1 2 4

(V) *Design of Channels*—When designing a new canal system, the *culturable* area of the tract to be irrigated should first be ascertained, and then the proportions of that area to be irrigated annually should be decided This may vary between 20 and 80 per cent, according to the amount of water available, or the risk, if any, there may be of waterlogging the soil by excessive irrigation Next, of the annually irrigable area it should be decided how much to irrigate in the *kharif*, and how much in the *rabi* season The usual proportions are $K = R$, or $2K = R$ If the winter water supply be scanty, or if there be serious risk of waterlogging, the former ratio is preferable, the more especially because the summer flow contains more fertilising silt than that of the winter, but $K = R$ means canal capacities 50 per cent greater (and more expensive) than $2K = R$ Thus, suppose we have to irrigate 30,000 acres annually

If $K = R = 15{,}000$ acres, and "Duty" = kharif 100, rabi 200, we require Average Supply Kharif 150, and Rabi 75 cusecs

But if $K = 10{,}000$, $R = 20{,}000$ acres, then average supply = $K = R$ = 100 cusecs And full capacity is 25 per cent greater than kharif average supply The capacities of a Rajbaha should be reckoned, reach by reach, from its tail, upstreamwards, by dividing the kharif area to be irrigated from each reach by the kharif F S F of 80 acres per cusec, and adding up the volumes thus determined

CHAPTER XIV

SCREW GEAR FOR TANK SLUICES, AND ROLLER GATES

(1) It is proposed in this chapter to give a short account of the best type of screw lifting gear suitable for tank sluices, as also of different types of rollers in use in vertical draw sluice gates

Figs 1 to 4 and Fig 7 are rough sketches illustrative of the principle of different systems of screw lifting gear In Fig 1 the motive power is applied to a female screw cut in the base of the handle itself On revolving the handle the solid screw, rising, lifts the gate, but there is no means of forcing

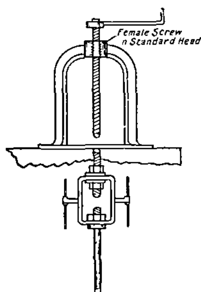


Fig 1

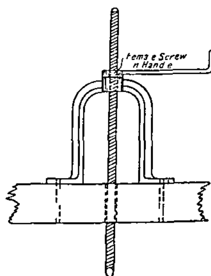


Fig 2

the gate down, unless its dead weight is sufficient for the purpose The author has actually seen this extremely primitive gear applied to wooden sluices of Government tanks in Burma and was privileged to witness the operation of lowering the gate against a head of water

The procedure adopted was for one cultivator to stand and jump on the gate while his companion belaboured the top of the screw rod with a large stone, the handle having been previously run up to the top The screw rod it may be added, was fixed to the gate at its lower extremity and so could not itself revolve

(2) In Fig 2 the motive power is applied to the screw rod, the female screw being cut in the head of the standard In this case the screw rod itself revolves and consequently has to be provided with a dog chain swivel joint

either at the gate fastening or preferably as near the screw head as possibl thus reducing the torsional stress on the rod The handle rises and falls wi the rod to the extremity of which it is keyed

This type is a great advance on the last and is quite suitable for mo plugs which are only lifted 2 feet at the outside The swivel should l prevented from turning by two arms which slide up and down guide roc This is shown roughly in the sketch

(3) Figs 3 4 and 5 represent the ordinarily adopted lifting gear in whi the female screw is revolved The solid screw rod connected with the ga

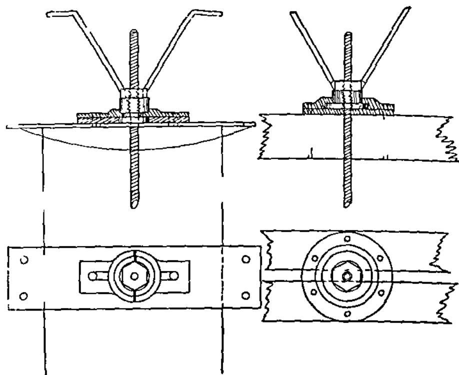


Fig 3 Fig 4

passes through the former and rises above the platform This arrangeme is on the same principle as Fig 1 only the female screw itself is prevent from rising by the use of a trust plate which either works in an annul groove cast in the brass screw head as in Fig 3 or itself is provided with holding down as well as a ba e plate which embrace a circular collar cast screwed on to the head piece as in Fig 4

In Fig 3 the steel or iron plate is in two halves and each half is provid with a slot as shown in plan This enables the two halves of the thrust pla to be disengaged and removed by slackening the fastening nuts and pushi the plates backward the bolts traversing the slot The author has se several of these employed in lifting tank sluices but they proved very ha to work In Fig 4 the holding down upper plate is in one piece the fe female screw being provided in this ca e with a collar at its base Fig 5

taken from the Madras Manual and exemplifies the usual Madras practice where screws are very commonly employed In this case the thrust plate also is in one piece and is slipped over the projecting shoulder of the

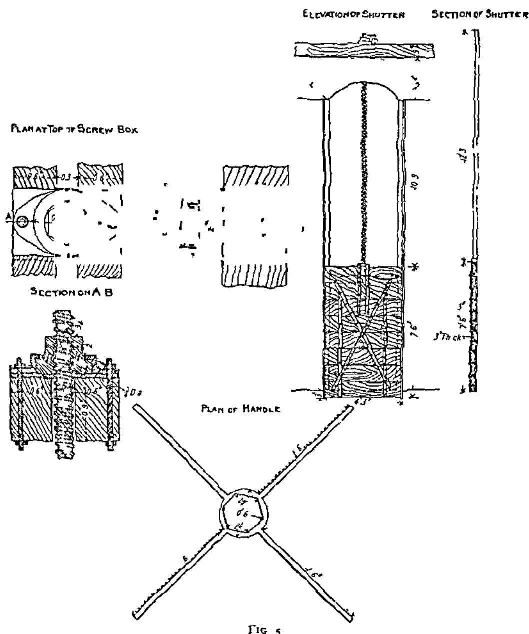


FIG 5

brass screw head and bolted in position If this thrust plate is truly cast and planed it should be as good as if not preferable to the form illustrated in Fig 3

(4) Fig 6 represents another development of the same principle and is used in Madras for lifting a series of grates one above another which close the larger openings of weir sluices or head regulators As already men

tioned in previous chapters this style of lifting gear for such purposes is now completely obsolete The introduction of anti friction rollers enables gates when released to fall into position by their own weight without requiring the application of any power to force them down against a head of water Hence they can be lifted by a travelling winch an arrangement much more economical than the lines of long expensive screw rods shown in Fig 6 In this gear the female screw is formed at the end of a long pipe, at the top of which pipe is a solid head to which the power handle is applied and which is held fixed by a thrust plate As the pipe is revolved to raise the gate the solid rod attached to the latter is drawn up inside the pipe being thus protected from dust or water

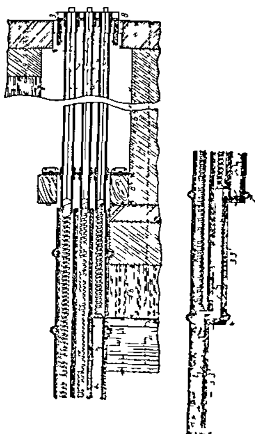


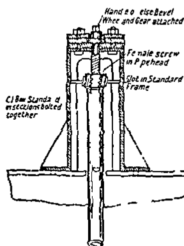
FIG 6

As screws in future are never likely to be employed except for very short lifts such as those of small deep reservoir sluices of which the outside lift is 3 feet it is quite clear that the principle of the motive power being applied to the short hollow screw thus involving a long solid screw rod subjected to compressional and torsional stress is not at all economical

(6) The arrangement illustrated in Fig 7 is undoubtedly far superior to the former In this the power is applied to the *male* screw which can be quite short a little over the lift in length This should be of solid cast steel As shown in the illustration the thrust collar and plates are situated at the rod head and consist of three plates the upper the distance plate and the base plate The screw rod is threaded through the base plate while the upper plates are superimposed and the whole bolted through as shown on plan (Fig 7) The upper and lower plates should be of brass the middle distance plate which is subjected to no strain of friction being of planed iron or else the middle and top plate could be combined in one piece of brass as in Fig 4 The solid screw rod passes into a pipe in the head of which is the female screw This should be of brass and in some cases the whole pipe is made of brass but this seems a needless extravagance The pipe head is held rigid by two arms running in guide slots cut in the frame of the standard Thus the

torsional stress is at once absorbed, and the position of the ends of the arms, one of which can be made to project beyond the frame, indicate, by means of a pointer on a graduated scale, the exact height at which the gate stands above its sill The pipe is rigidly connected with the gate, and from its ring section is clearly much better suited to withstand compression than a solid rod of greater weight of metal It can also be made of ordinary gas or water piping The introduction of ball bearings above and below the thrust discs would be a further improvement

(6) This system for short lifts is by far the best of any other illustrated in these figures It is remarkable that it is not given in the ' Madras Manual," all the screw lifting gear in which are of the other types, in which the motive power is applied to the female screw The only tank sluice fitted with this gear seen by the author was that in Kalawewa Tank, in Ceylon In this a hand wheel turned a worm, or archimedean screw, which engaged a bevel wheel keyed onto the top of the shaft The sluice gate in this case was 3 feet square under a head of 25 feet, and yet the gate was manipulated with the greatest ease by one man



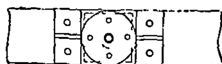


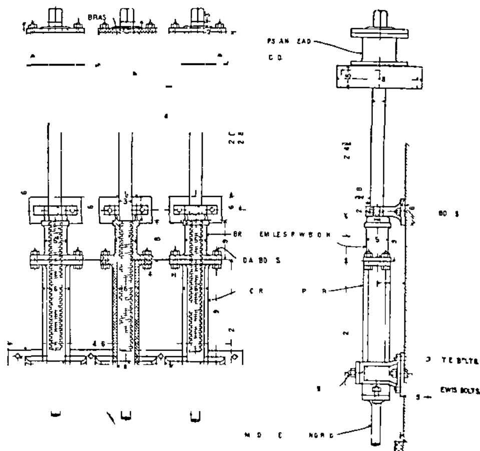
Fig 7

The screw heads or standards, can be supported on wooden beams or a cast-iron fish-bellied girder with double webs Two rolled beams connected by a plate bolted to their upper flanges would suit equally well, the lifting rod passing between their webs

(7) Fig 8 represents a group of three screws which operate the pipe valves in the Waghad Tank outlet culvert which are shown in Fig 1, Chap XII These are on the principle of Fig 7 At the top is a thrust box containing brasses with hollow annular ways in which similar annular projections on the steel screw rod work Below is a hollow square pillar in the head of which is a female screw, engaged by the screw of the lifting rod To the base of the pillar, which is moved up and down through a square bracket bearing the lifting rod of the gate is attached

(8) Fig 9 is on the same good principle, but in an improved style Here the lifting screw works against a step at its base and is provided with a thrust collar working in a brass thrust box at the top It is therefore practically independent of any deviation from the exact alignment of the gate lifting rod, and is thus not subjected to other legitimate stresses The lifting screw is enclosed in a round cast-iron standard, in the hollow pipe of

which a circular brass stud is free to move In this is cut the female screw and it is prevented from revolving by projecting arms which run in two slots cut in the side of the box as in Fig 7 To these projections two links are attached which latter below the frame are joined by a cross bar or pin on which the gate lifting rod is keyed Instead of the clumsy capstan bar a bevel wheel could be keyed on the top of the screw rod and a bracket fitted to the frame or cast on it would carry a pinion and hand wheel The



F S —L ft Scre Gear of Wagl ad Tank

hand wheel could be made removable to prevent unauthori ed t mpering with the sluices Ball bearings hould al o be arranged for at the thrust collars

(9) An example of American lifting gear taken from Wilson s Irri atio n Engineering is given in Fig 10 Here a bevel wheel i k yed on the female screw block which is held in the jaw of the ca t iron fram carrying the pinion wheel and han l wheel

Steel ball bearing di cs are fitted at the ba of th e femal scr w bl ck where it bears again t the fram Th lifting cre v r d i squar in ecti n below the screw and pas thr u h a square h l in a pr jection at the base of the cast iron frame

This arrangement is obviously inferior to that of the two last examples in that the screw is exposed in the open to dust and rain and when the sluice gates are open sticks up in an absurd manner as if to invite damage whereas in the other case the screw is protected from injury of any kind being enclosed in a box out of sight Length of rod is also saved This apparatus or one in which a pinion engages a rack appears to be universal in the

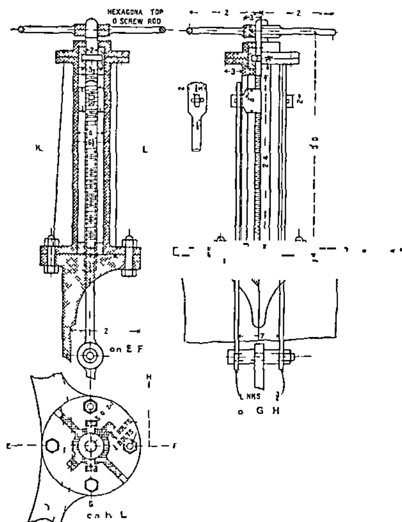


Fig 9— Battman s Screw Lift Gear

States The adoption of ball or roller bearings at the points of thrust is worthy of imitation

(10) A very instructive example is given in Figs 11 11a and 11b of the screw lifting gear of the Bhatgarh Dam undersluices a section of which is given in Fig 15 Chap II and for which we are indebted to The Irrigation Works of India These sluice gates have a lift of 8 feet under a mean head of 80 feet the area of the sluice ways being 8 feet by 4 feet the pressure on

each is therefore $r\,h\,A = \frac{1}{36} \times 80 \times 4 \times 8 = 71$ tons The screw rod is 5¼ inches diameter for the screwed upper length the remainder being 5¼ inches square These heavy expensive forged rods are about 87 feet in total length though made up in 12½ feet lengths and are supported at intervals against buckling by passing through square holes in brackets projecting from the rear face of the dam

The motive power is as usual applied to the brass female screw which is provided outside with two collars which work in corresponding grooves in a thrust box This latter which is of cast iron is in two pieces bolted together horizontally and further provided with the usual vertical holding

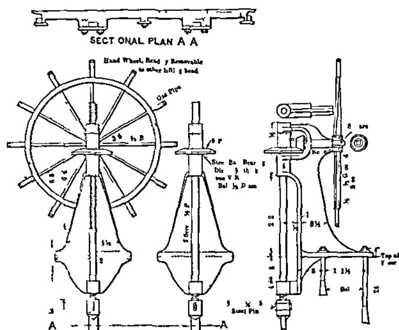


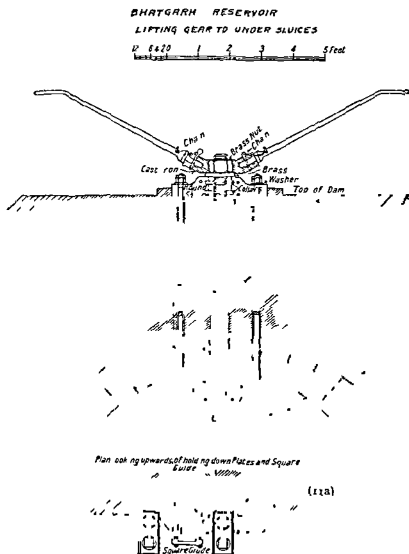
FIG 10

down bolts (*vide* plan of lifting nut collar etc in Fig 11a) The squared part of the rod commences at the termination of the screwed upper part which has a run of 8 feet and this passes through a square brass guide which prevents the rod revolving and relieves it of any torsional strain below this point The screw lifting rod rises through the capstan nut and when the gate is fully open must stick out some 9 feet above the platform level on top of the dam

(11) This manifestly unsuitable arrangement could easily be avoided by adopting the general design of Fig 7 8 or 9 viz of applying the motive power to the male screw which latter being provided with a collar working in a thrust box is prevented from vertical movement but not from revolving The female screw would be placed in the head of a long cast iron pipe which is fastened to the gate and which is prevented from revolving by attached arms the extremities of which slide in vertical grooves and is of course free to move vertically in either sense It is quite evident that this

arrangement would be much less expensive and equally effective as that portrayed in Fig 11

A further improvement would be to apply bevel gearing to the screw head in lieu of the capstan, which has to be worked by several men A bevel spur wheel could be fixed horizontally on the rod head, worked by one



Figs 11 11a

or two bevelled pinions turned by wheel handles revolving vertically, and anti friction rollers above and below the thrust collars

(12) The principle of the application of free rollers and roller wheels fixed on axles to draw gates, together with the devices employed in rendering the gate watertight will now be briefly noticed

When a gate or shutter slides in a groove, the pressure of the water forces it against the inner side of the vertical grooves, as also against the

lintel at the top and the cill at the bottom thus effectually preventing any leakage on all four sides of the shutter The resistance of the friction of the end side of the gate against that of the grooves is very considerable and as the parts in contact cannot be lubricated and the gate often remains unmoved for a long time the parts become rusty and the friction is thus increased greatly in excess of what would normally be the case Now the coefficient of friction of smooth metal surfaces in contact unlubricated is about 2 Thus if a gate were subjected to a horizontal water pressure of 1 000 lbs it could be lifted neglecting its own weight by a force of 1 000 × 2 — 200 lbs On the other hand when the surfaces are not smooth the coefficient of friction may possibly become as great as unity Hence a gate without rollers requires power not only to lift it but to force it down as its own submerged weight is insufficient to overcome the friction induced by the pressure of the water This is the reason why screw power has hitherto been so largely used as lifting apparatus

By the simple device of mounting the gate on wheels with axle bearings the friction is greatly diminished so that the requisite lifting power is considerably reduced and none is required to lower the gate that being effected by its own weight

The combined axle and roller friction now induced which is similar to that of a railway truck can be taken as one eightieth of the water pressure plus the weight of the gate itself in the process of lifting Thus the net lifting power required supposing the water pressure to be 1 000 lbs would be $\frac{1\,000}{80} = (12\tfrac{1}{2} + W)$ lbs

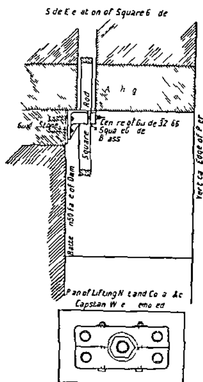


FIG 11b

If the axle bearings are not lubricated the coefficient of friction will naturally be much greater Within certain limits a roller wheel of large diameter would give better results than one of smaller size The rolling friction induced between the wheel rim and the surface would be less in the former case and in addition the leverage of the greater wheel radius acts in favour of the motive power

(13) When a roller or wheel is used with a gate there is naturally no close contact between the side of the gate and that of the sluice opening or of the table of the groove the gate being raised off it In order to prevent leakage through this opening the device of stanching rods is usually adopted which consists generally of a round solid rod or else gas piping which is fastened to the gate at the top only the rest hanging free The pressure of the water

forces this against the opening slit existing between the gate and the jamb, or shoulder of the masonry, forming a perfect watertight closure When the gate is pulled up or down the stanching rod goes with it, scraping against the jamb , the friction thus set up is quite negligible

With Indian roller gates stanching rods do not appear to be employed A watertight connection is formed by either slightly inclining the roller path or else the gate, so that when the gate is fully down close connection is formed In India stanching is commonly effected by means of basketfuls of stable, or cow-shed litter applied up stream of the leaky passage It is not an elegant mechanical device, but effectively staunch

(14) Gate rollers or wheels consist either of small solid rollers mounted on axles which pass through axle boxes fixed to the gate The method formerly used in the Chenab weir-sluice gates is exhibited in Fig 12

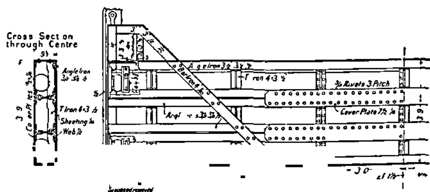


FIG 12 —Middle Gate of Chenab Weir Sluices

Recently this type of gate has been scrapped and "Stoney" pattern live-roller gates, in one piece, substituted

In other sluice gates the rollers consist of a pair of trolley wheels with long axle bars reaching across with the wheels keyed on, as in the case of a railway truck The large regulating gates across the Rhone at Geneva which are 25 × 10 or × 15, are mounted on quite large wheels keyed on shafts, the wheels being double flanged and working on an ordinary vertical steel rail fixed upright against the jambs or shoulders of the sluice opening

In Indian and Egyptian works, draw gates usually work in cast-iron grooves , the grooves protect the lifting chains from the current and afford a recess in which the chains hang safely when the gates are in place, and are disconnected from the travelling winch

A great improvement would be to adopt forced axle lubrication as is done to ' Stoney ' gate rollers by flexible tubes connecting with the surface The journals could then be packed in watertight stuffing boxes like the piston of a stream cylinder

(15) We have hitherto considered the combined axle and rolling friction which is induced by small wheels fixed on axles running in bearings bolted to

the gate When, however, the pressure is very considerable, due to a great head of water or the exceptional dimensions of the opening, another arrangement, viz, that of free rollers, is made use of These rollers run on axles, but there is no pressure induced on the latter, the axles are only used to keep the rollers the proper distance apart, and are fixed in a frame This roller cradle is suspended in the groove unattached to the gate As the side of the gate bears against the rollers, when the former is moved the rollers revolve, and the whole frame of rollers rises, and also falls, with the gate The friction induced is pure roller friction, which, between smooth surfaces, is something very small indeed, so much so that its coefficient can be entirely ignored, the lifting power being only the weight of the gate plus the friction of the lifting apparatus Again, under suitable conditions, the gate can be counterpoised by weights hanging from overhead pulleys, and in such cases gates of the largest size can be manipulated by one man These free rollers are termed Stoney's Patent Anti friction Rollers"

The diagrams in Fig 13 represent a sluice gate fitted with free rollers in sectional plan and elevation It will be seen that the frame of the gate on plan just clears the sluice wall and projects beyond the groove These spaces are closed by stanching rods These are simple round rods of about 2 inches diameter which are fastened to the gate, but have lateral play, so that when the gate is under pressure they are forced into the corner, effectually closing the aperture On the gate being raised they are carried out with it, still bearing against the side of the gate and the sluice wall, the friction thus induced is however, quite trifling Owing to the suspending pulleys the roller frame moves half as fast as the gate

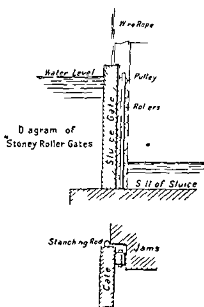


FIG 13

(16) In Fig 14, which is derived from Vol CLII, "Min Pro Inst C E," the details of the roller gates used in the sluices of the Assuan Dam viz, those at R L 100 00, are given The following is the description by Mr F W Scott Stokes, M Inst C E, the author of the paper on the sluices and lock gates of the Assuan Dam

(Sluices at R L 96 and R L 100) (Fig 14)

"The culverts are 2 metres (6 feet 6¼ inches) wide by 3 5 metres (11 feet 6 inches) high The entrance to the culvert is well mouthed and is roofed over by a casting curved on the dam face, and flat at the place where it joins the sluice lintel The grooves, lintel, and cill of the sluice are of cast iron

arranged with machine faces, and put together with turned bolts The sluice is built up of a steel plate skin with rolled strengthening girders at the back, framed into cast iron beams on each side, which form the roller paths Adjustable bars are fitted on each side of the face to reduce the leakage, as wear takes place owing to the cutting action of the silt in the water On the top of the sluice is also fitted an adjustable bar, which shuts down on the lintel at the same moment that the bottom of the skin lands on the cill, and

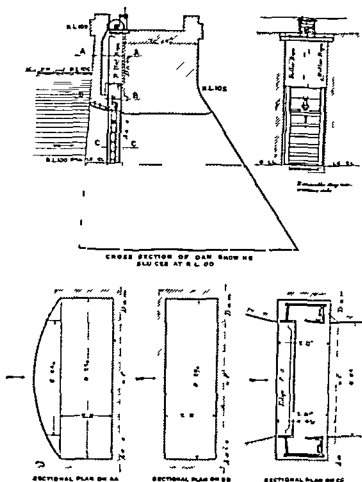


FIG 14—Assuan Dam Sluices at R L 100·0

thus makes a watertight joint The rollers are arranged in cradles formed of flat bars and are hung in position on each side of the gates by steel wire ropes The crab has two speeds of lift, namely, for small adjustments by one man, or for quick working by four An automatic self sustaining gear is provided so that the sluice (which is not counterbalanced in any way) cannot run down, the handles having to be turned when lowering in order to release the gear The barrel shaft is worked by means of a worm wheel, and a worm fitted with a ball thrust bearing *All the bearings and pulleys are* fitted with screw down grease lubricators, this is the more advisable owing to the sand and dust

(Sluices with Rollers at R L 92) (Fig 15)

(17) " These twenty-five sluices are 7 metres (23 feet) high by 2 metres (6 feet 6¾ inches) wide, and are for the purpose of regulation when the fifty without rollers have been lowered, and before the water rises high enough for those at R L 96 and R L 100 to give the required discharge

" As the head above cill level is 14 metres (45 feet 11 inches) the rollers

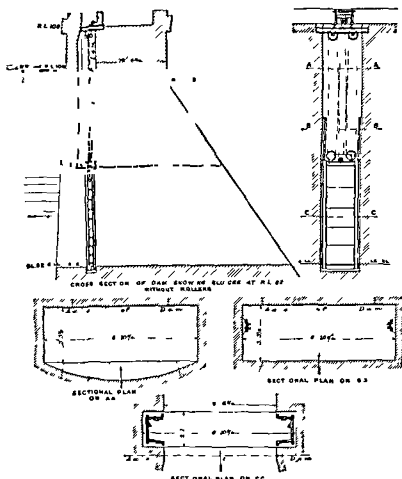


Fig 15 —Assuan Dam Sluices at R L 92 00

and other parts have to be of much more ample proportions and the grooves built into the masonry of considerably larger size than those of the sluices of R L 100 and R L 96 The action of the water passing through the culvert at a high velocity has also to be avoided hence the rollers are kept well back from the face, and an inclined shield plate is provided to protect them from a direct flow A whirlpool motion is set up in the grooves, but it is not of sufficient force to disturb the rollers hanging below the bottom of the gate when raised The grooves are built of four sections weighing about 2 tons each They are bolted together with turned bolts, and are arranged with projections on each side to key into the masonry and prevent

the leakage of water round the back The sluice is built of heavy specially rolled steel joists with steel skin plates and cast iron roller path guiders on each side

A vertical rod which is contained in a groove bolted to the skin on each side of the face is pressed by the water into the corner formed by the face

FIG 16—Assuan Dam Sluices at R L 87 50

of the sluice and the face of the fixed frame thus making a watertight joint

(Sluices at R L 87 50) *(Fig 16)*

(18) It was originally intended to have the lowest sluices in the dam at R L 84 but it was found that R L 87 50 would suit the surface levels and channels better The head of water above cill level is 18 5 metres (60 feet 8 inches) giving a pressure of about 210 tons against the sluice The entrance to the culvert is bell mouthed and roofed by an arched casting A slight drop is arranged on the down stream side of the sluice cill which has been found to reduce greatly the wear and tear on the culvert bottom

and to give a freer discharge under the sluice A similar drop has been given to the other culverts The grooves are built of four sections, bolted together and to the lintel and sill castings Stanching rods are fitted to the sluice, as in those for R L 92 with rollers The gate is built of special rolled steel joists framed into two cast roller paths The skin is riveted to the joists in sections, which, when in position, are finally fixed together by turned bolts The sluice gate is carried by a steel wire rope in two parts arranged round pulleys on the gate and on girders at the pit top Forced grease lubrication is employed as in other sluices and gear The crab is similar in design to those for the sluices at R L 96 and 100"

(19) Fig 17 illustrates the case of double roller gates as used for undersluices of wide span The outer gate is generally the lower one These gates require a watertight joint between them when both are lowered The projection of the rollers and the thickness of the middle groove flanges places them somewhat far apart This difficulty can be overcome by projecting the top plate of the lower gate inwards, so that a narrow flat plate will cover the space, or else a plate attached to and suitably hinged to the base of the inner upper gate, so that when the lower is raised it will lift the flap, which will still keep in contact with some projecting vertical bars inserted for the purpose and thus either gate can be lowered to the floor

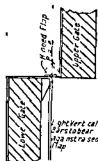


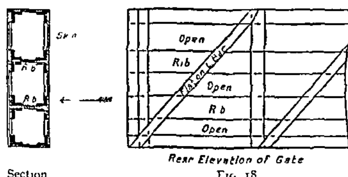
Fig 17

Roller gates are best designed with a shoulder, or else an angle iron can project to form an abutting line for the vertical stanching rods If necessary, stanching rods can be applied inside the groove itself at the ends of the gates

(20) Gates are usually built up as plate girders, but sometimes rolled beams are used for the longitudinals, as in the Assuan Dam sluices The gate consists of the outer skin and a series of longitudinal ribs which convey the horizontal water pressure to the rollers As the pressure is greatest at the base of the gate, the ribs should be closer together near the lower end of the gate, the spacing being gradually increased so as to cause each rib to bear approximately the same horizontal pressure The first point to be determined is the depth of the ribs, *i e*, the width of the gate at the centre This may vary from $\frac{1}{20}$ to $\frac{1}{10}$ of the span Representing the distance apart of the lowest ribs as b, and the head of water above the middle point between the ribs as h, and the length between the piers or span as S, the pressure to be supported by each rib will be $\frac{1}{2}\,u\,(h \times S \times b) = W$ in feet tons, u being $\frac{1}{35}$ ton, the weight of a cubic foot of water

This pressure W is a distributed load, consequently the bending moment M will be $\frac{WL}{8}$, L being the distance between the centre of the roller supports which is somewhat greater than S The rib in ordinary cases will be

of double T section formed of two angle irons, the upper tables of which are riveted to the skin and the lower to the web If the skin plate on one side and the web be neglected from consideration, the sectional area of each of



Section Fig 18

the flanges will consist of that of the angle iron plus the portion of plate enclosed between them, as shown shaded in Fig 18

(21) The effective depth of the girder can then be considered to be not the extreme depth of the double T beam, but the distance between the centre of gravity of the angle irons or d, which lines fall a little below the inner side of the top tables of the angle irons (*vide* Fig 18) Tensile and compressive stress on the top and bottom flanges will be $\frac{WL}{8d}$ tons The safe stress on iron may be taken as 4½ tons, and that on steel as 6 tons per square inch, termed m The required net sectional area of the flanges, less rivet holes, will be $a = \frac{WL}{8dm}$ If rolled beams are used, the thickness of the web will have to be taken into consideration, and the moment of inertia about the neutral axis of the beam calculated (*vide* "Molesworth's Pocket Book," p 130) Then $R = \frac{mI}{N}$, in which R is the resistance of the beam to flexure which should equal $\frac{WL}{8}$, m the safe compressive or tensile stress per square inch of the metal, I the moment of inertia, and N in this case $= 2, \frac{D}{2}$ being the outside depth of the section

(22) No continuous plate is required on the inside of the gate, but *vertical and inclined strips of angle or flat bar at intervals*, so as to enable access to be obtained for painting the parts The area of the skin plate, as also that of these strips reduced to the equivalent of a thinner plate over the whole surface, could be included in the sectional area of the flanges, or in the value obtained for I The webs of the ribs will naturally have to be strengthened by angle or T irons at intervals, as is usual in plate girders In large single gates, where space is not confined, the ribs are generally composed of open lattice work with a good curvature to the centre, *similar* to a

bow-string girder placed on its side The top and bottom of the gate should be plated over

(23) The shearing stress on the axles of the gate roller wheels, if such be adopted, will be one quarter the total water pressure on the whole gate, *i.e*, if two sets of rollers be used The position of the rollers should be fixed with reference to the pressure so as to secure the upper having the same pressure as the lower roller

The axles, if supported on both sides of the rollers, are subjected to double shear, consequently their sectional area should be $\frac{W}{8m}$, W being the total water pressure on the whole gate $= wh \times ld$, h being the mean head and ld the area of gate, and m the safe shearing stress of the metal, viz, 5 tons for iron and $6\frac{1}{2}$ tons for cast steel

It might be further noted that the web of the ribs is subjected to shearing stress which is greatest at the ends of the gate, where it will amount to half the total water pressure on the gate, presumably it is sufficiently supported against buckling Its effective sectional area should equal this pressure in tons divided by m the safe shearing stress of the metal, generally assumed as the same or less than the compressive strength The cover plates for joints in the skin should be of T section These are not shown in the sketch in Fig 18 The gate is not only subject to transverse thrust but to vertical bending moment due to its own weight This is met by the disposition of the bars on the rear side, as shown in the elevation, and by the skin on the face of the gate



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